California High-Speed Train Project

Agreement No.: HSR 13-06
Book 3, Part E, Subpart 5

Geotechnical Baseline Report West Clinton Avenue to East American Avenue

HSR 13-06 - EXECUTION VERSION

Prepared for
CALIFORNIA High-Speed Rail Authority

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CALIFORNIA HIGH-SPEED TRAIN

Engineering Report

RECORD SET PRELIMINARY
ENGINEERING FOR PROCUREMENT

Fresno to Bakersfield

Sierra Subdivision
Procurement Package 1
Geotechnical Baseline
Report for Bid

November 2012

HSR 13-06 - EXECUTION VERSION

CALIFORNIA
High-Speed Rail Authority

RECORD SET
SIERRA SUBDIVISION
CONTRACT PACKAGE 1
GEOTECHNICAL BASELINE REPORT
FOR BID

HSR 13-06 - EXECUTION VERSION

Prepared by:

URS/HMM/Arup Joint Venture

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Record Set Preliminary Engineering for Procurement

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This report has been prepared under the direction of the following registered Geotechnical Engineer.

Martin Walker, PE, GE

November 1, 2012
Date

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ABBREVIATIONS

AASHTO	American Association of State Highway Transportation Officials
ADS	Anti-Drag System
API	American Petroleum Institute
ASTM	ASTM International (formerly American Society for Testing and Materials)
Authority	California High-Speed Rail Authority
BGL	Below Ground Level
Caltrans	California Department of Transportation
CIDH	Cast-in-Drilled-Hole
cm	Centimeter
CP1	Contract Package 1
CPT	Cone Penetration Test
EIS/EIR	Environmental Impact Statement/Report
E_s	Soil Modulus
F-B	Fresno to Bakersfield
ft	Feet
g	Gravity
GBR-B	Geotechnical Baseline Report for Bid
GBR-C	Geotechnical Baseline Report for Construction
GDR	Geotechnical Data Report
HMM	Hatch Mott MacDonald
HST	California High-Speed Train Project
JV	HMM/URS/Arup Joint Venture
k_h	Modulus of Horizontal Subgrade Reaction
k'_v	Modulus of Vertical Subgrade Reaction
kN	Kilonewton
MCE	Maximum Considered Earthquake
mi	Miles
mm	Millimeters
M_W	Moment Magnitude
$(N_1)_{60}$	Standard Penetration Test N-Value Corrected for Overburden and Field Procedures
N_{60}	Standard Penetration Test N-Value Corrected for Hammer Energy
NA	Not Available
NAVD88	1988 North American Vertical Datum
NPDES	National Pollutant Discharge Elimination System
NRCS	National Resources Conservation Service
OBE	Operating Basis Earthquake
OSHA	Occupational Safety and Health Administration
ppm	Parts Per Million
PEP	Preliminary Engineering for Procurement
q_c	CPT Cone Resistance
q_t	CPT Cone Resistance Corrected for Pore Water Effects
SBT_N	Normalized CPT Soil Behavior Type
sec	second
SJV	San Joaquin Valley
SJVRR	San Joaquin Valley Railroad
SPT	Standard Penetration Test
SR	State Route
SWPPP	Storm Water Pollution Prevention Plan
T	Period
TM	Technical Memorandum
umhos	Micromhos

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UPRR	Union Pacific Railroad
USCS	Unified Soil Classification System
USDA	United States Department of Agriculture
USGS	United States Geological Survey
V_{s30}	Average Shear Wave Velocity in the Upper 30 Meters
WEAP	Wave Equation Analysis of Piles

1.0 Introduction

The California High-Speed Train (HST) Project will provide intercity, high-speed train service throughout California's major population centers. A joint venture (JV) between URS, Hatch Mott MacDonald (HMM), and Arup has been contracted by the California High-Speed Rail Authority (Authority) to perform preliminary engineering for procurement (PEP) engineering services for the portion of the project that extends between Fresno and Bakersfield.

Contract Package 1 (CP1) of the California HST Project extends from Avenue 17 in Madera to E American Avenue in Fresno. The northern section of CP1, from Avenue 17 to W Clinton Avenue is within the Merced to Fresno segment of the HST. The southern segment of CP1, from W Clinton Avenue to about E American Avenue, is within the Fresno to Bakersfield (F-B) segment of the HST. The JV is responsible for this segment.

F-B CP1 corridor spans approximately 9 miles from W Clinton Avenue to about E American Avenue in Fresno County, as shown in Figure 1.1-1. For brevity, where CP1 is referred to in this report, it shall be construed to mean only the F-B section of the corridor contracted to the JV.

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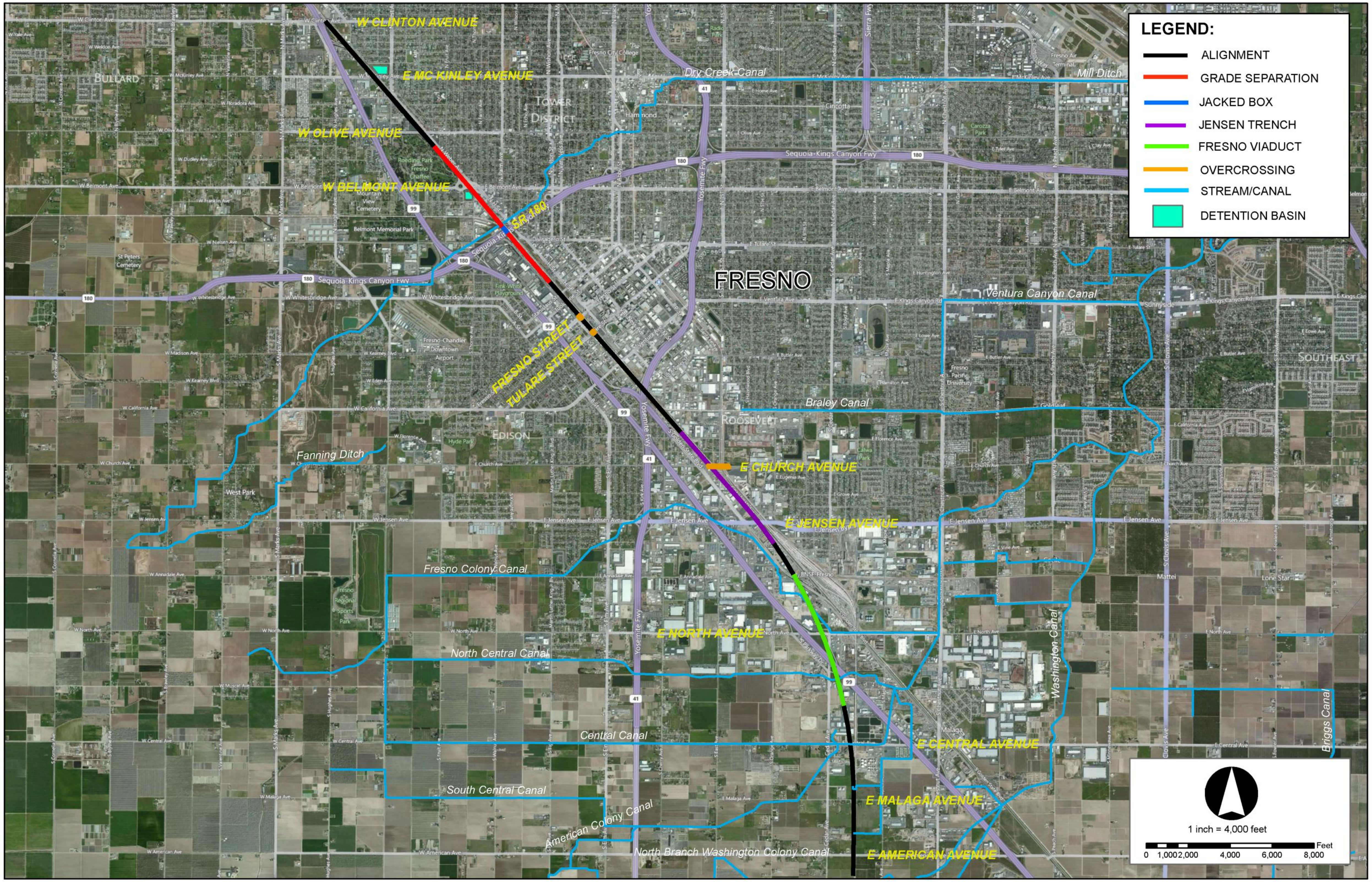

Figure 1.1-1
Site Vicinity Map

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1.1 Geotechnical Contract Documents

The geotechnical Contract Documents include this Fresno to Bakersfield Geotechnical Baseline Report for Bid (GBR-B) and the Fresno to Bakersfield Geotechnical Data Report (GDR) CP1 (URS/HMM/Arup 2012). The GDR provides details of the geotechnical investigation such as drilling procedures, soil sampling, in situ testing, hydrogeologic testing, and historical geotechnical information gathered prior to the exploration phase. The GDR also includes exploration logs, details pertaining to laboratory testing, procedures used to conduct various index tests, strength and deformation tests, test results and a limited environmental assessment. Definitions for terms used in both the GBR-B and GDR are contained in the Glossary.

Contractually, this GBR-B and the referenced GDR cover only the F-B CP1 corridor.

1.2 Purpose

The principal purpose of this GBR-B is to set baselines for conditions to facilitate the bidding process such that all bidders can rely on a single contractual interpretation of the geotechnical conditions when preparing their bids. This report summarizes the geotechnical basis for PEP and anticipated conditions for construction of the CP1 alignment, which extends between W Clinton Avenue and about E American Avenue.

This GBR-B is a representation of the conditions upon which the Contractor may rely for bidding. Geotechnical investigations conducted in preparation of the GDR are considered preliminary and should not be solely relied on for final design. It is incumbent upon the Contractor to conduct supplemental investigations adequate to complete final design and prepare a Geotechnical Baseline Report for Construction (GBR-C). The GBR-C will serve as the basis of resolution for differing site conditions during construction. The GBR-B has been prepared such that it will be superseded by the GBR-C, and the GBR-C will incorporate additional geotechnical exploration data and analyses. The GBR-C will become the basis of final design and construction conditions.

The engineering judgment applied in the interpolations and extrapolations of information contained in the GDR reflect the view of the Authority in establishing the baseline conditions. The baseline conditions presented in this report will (1) serve as a baseline for geotechnical conditions anticipated to be encountered and (2) assist the contractor in evaluating the requirements for excavating and supporting the ground.

1.3 Report Structure

This report has been prepared in general accordance with Technical Memorandum (TM) 2.9.2 Geotechnical Reports Preparations Guidelines and the latest edition of the American Society of Civil Engineers' publication *Geotechnical Baseline Reports for Construction – Suggested Guidelines* (Essex 2007). Sections 1 through 5 provide background information while Sections 6 through 9 provide specific recommendations related to ground characterization and behavior. Sections 10 and 11 provide reference information.

Section 1 provides an introduction to the project including project location, report purpose and organization. Section 2 provides a project description including key project features and existing man-made structures of significance to the project. Section 3 describes sources of geotechnical information including prior geotechnical reports, TMs, data from desk studies and data from the PEP Geotechnical Investigation for CP1. Section 4 describes the project setting through physiography, geology, seismicity, and hydrogeology; Section 5 describes previous construction experience in the project vicinity.

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Section 6 presents ground characterization and geotechnical baselines; Section 7 describes design considerations for the various proposed structures; Section 8 describes construction considerations; Section 9 describes recommended instrumentation and monitoring during construction.

Section 10 is a list of documents referenced in this report; Section 11 is a glossary of terms used in this report.

1.4 Basis of Report

The baseline values in this report have been developed from geotechnical information and data gathered through desk studies and the PEP Geotechnical Investigation, which included widely spaced exploratory boreholes, cone penetration tests (CPTs), and laboratory and field tests. The results from this investigation are summarized in the GDR.

1.5 Project Constraints and Restrictions

The baseline recommendations in this report have been derived from the available data. Limited site access, limited historical data, and wide spacing of explorations constrain the recommendations to a level appropriate for PEP, not final design. For the portion of the F-B CP1 between W. Clinton Avenue and Golden State Boulevard, preliminary engineering for procurement (PEP) for structures was advanced using geotechnical parameters from historical data only. When the GDR and this GBR-B became available, design assumptions were checked against the baselines herein for applicability of the parameters developed with historical data. The design of structures south of Golden State Boulevard was advanced using the baseline geotechnical parameters presented herein.

During construction, ground behavior will be influenced by the Contractor's selected design, equipment, means, methods, and level of workmanship. The Contractor must assess how these factors will influence ground behavior and baseline values provided in this report in consideration of the project as a whole.

The baseline configuration for CP1 is shown in the Contract Plans and Specifications (Contract Documents). Certain construction elements in the Contract Documents are mandatory, while others are the Contractor's responsibility to develop. The mandatory requirements are defined in the Contract Documents. The site conditions described herein are intended to apply to the reference design in the Contract Documents.

2.0 Project Description

The F-B CP1 project alignment starts at the intersection of W Clinton Avenue and N Golden State Boulevard in Fresno, California. The alignment continues southeast along Golden State Boulevard for about 2 miles to W Belmont Avenue. South of W Belmont Avenue, the alignment continues southeast between an existing rail right-of-way and G Street for about 2.8 miles where G Street terminates at Golden State Boulevard. At this point, the alignment continues southeast about 1.1 miles between the existing rail right-of-way and Golden State Boulevard until its intersection with E Jensen Avenue, where it veers south crossing Golden State Boulevard, E North Avenue, Golden State Freeway, and E Central Avenue. South of E Central Avenue, the alignment is adjacent to an existing rail right-of-way and continues south for about 2 miles until the end of CP1 just north of E American Avenue.

The alignment is adjacent to Roeding Park north of Belmont Avenue; crosses the Sequoia Kings Canyon Freeway (SR 180), Yosemite Freeway (SR 41), and Golden State Highway (SR 99); crosses irrigation canals north of SR 180 and south of E Central Avenue; and is adjacent to a detention basin at the intersection of E McKinley Avenue and N Golden State Boulevard, and at W Belmont Avenue.

Baseline configuration for CP1 includes at-grade and embankments rail sections, two trenches, a viaduct, and a jacked box tunnel. CP1 work also includes numerous secondary transverse vehicular and pedestrian bridges at select local street intersections. Shallow and deep foundations, retaining walls, and earthwork embankments will be required for the proposed improvements. The key project features are described in Table 2.0-1 from north to south.

Table 2.0-1
Significant Structures – CP1

Name	Physical Location	Approximate Size	Notes
At-Grade	Along N Golden State Boulevard. From W Clinton Avenue to 1,500 feet south of W Olive adjacent to Roeding Park	Width: 60 ft Length: 8,500 ft	Adjacent to detention basin at W McKinley Avenue Vehicular overcrossings proposed at W McKinley and W Olive Avenues
Fresno Grade Separation	From Roeding Park to about 1,200 feet southeast of El Dorado Street	Width: 60 ft Depth: 55 ft Length: 7,400 ft	Adjacent to Belmont Detention Basin Crosses under W Belmont Avenue, SJVRR spur, Dry Creek Canal, and SR 180 Jacked Box Tunnel proposed where alignment crosses SR 180
At-Grade	From about 1,200 feet southeast of El Dorado Street to about 400 feet southeast of E Church Avenue	Width: 100+ ft Length: 12,500 ft	HST overcrossings proposed at Fresno Street, Tulare Street and Ventura Avenue Pedestrian bridges proposed between Tuolumne and Stanislaus Streets, and at Ventura and E Church Avenues Vehicular overcrossings proposed at Tulare Street and Fresno Street Fresno Station planned in this reach of the alignment
Jensen Trench	From about 400 feet southeast of E Church Avenue to S Orange Avenue	Width: 100 ft Depth: 17 ft Length: 4,400 ft	Crosses under E Jensen Avenue Vehicular overcrossing proposed at E Church Avenue
Fresno Viaduct	From Golden State Boulevard to about 500 feet north of E Muscat Avenue	Width: 60 ft Height: 40 ft Length: 5,500 ft	Crosses Golden State Boulevard, E North Avenue, S Cedar Avenue, and SR 99
Embankment/ At-Grade	From about 500 feet north of E Muscat Avenue to E American Avenue	Width: 60 ft Length: 13,400 ft	Crosses irrigation canals adjacent to E Central Avenue, between E Malaga and E American Avenues, and between E Jefferson and E American Avenues Vehicular Overcrossings proposed at E American Avenue and E Central Avenue crossings

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3.0 Sources of Geologic and Geotechnical Information

3.1 Project Sources

Data and information for this report were primarily obtained from publically available reports and results of the 30% Geotechnical Investigation. Additional information was obtained from the following reference documents:

- F-B Draft EIS/EIR (URS/HMM/Arup 2012a)
- F-B Geology, Soils and Seismicity Technical Report (URS/HMM/Arup 2012b)
- F-B Geologic and Seismic Hazards Report (URS/HMM/Arup 2012c)
- F-B Water Hydrology, Hydraulics and Drainage Report (URS/HMM/Arup 2012d)
- F-B 15% Draft Utility Impact Report (URS/HMM/Arup 2012e)
- F-B Archeological Survey (URS/HMM/Arup 2011)

3.2 Site Investigations

The 30% Geotechnical Investigation for CP1 was conducted between October 10 and 28, 2011 and consisted of drilling 17 rotary-wash boreholes and performing 44 CPTs. Soil samples were collected from boreholes at 5-foot intervals using Standard Penetration Test (SPT) samplers driven with automatic hammers. Energy calibration tests were performed on the automatic hammers used during the exploration program.

In situ testing performed during the investigation included shear wave velocity profiles in 4 boreholes using the suspension velocity logging method, shear wave velocity profiles in 6 CPTs, and pore water pressure dissipation tests in 19 CPTs. Seven boreholes were converted to standpipe piezometers to monitor groundwater level fluctuations.

Laboratory testing was performed on representative soil samples to obtain index and engineering properties. Geotechnical index property testing included moisture content, grain-size analyses, Atterberg limits (plasticity), and organic content tests. Engineering property tests included direct shear strength, Modified Proctor compaction, California Bearing Ratio, and corrosion tests. Groundwater chemistry testing was also performed on water samples obtained from standpipe piezometers.

3.3 Historical Investigations

The primary source of publicly available historical geotechnical data collected during 15% design was from the California Department of Transportation (Caltrans) database of as-built construction records.

Caltrans data are mainly concentrated along Routes 41, 43, and 99, from projects dating between 1953 and 1997. For each project, several boreholes were drilled, logged, and plotted on a cross section. None of the Caltrans records contain laboratory test data. Borehole records collected from Caltrans extend to a maximum depth of 122 feet below ground level (BGL), with an average borehole depth of 42 feet BGL. Historical Caltrans data are included in Appendix A of the GDR.

4.0 Physiography & Geology Overview

The section provides a brief description of physiography, geology, and seismicity within the CP1 corridor. Detailed discussion of physiography, geology, and seismicity along the entire F-B alignment is presented in the Geologic and Seismic Hazards Report (URS/HMM/Arup 2012c).

4.1 Physiography

The CP1 alignment is located within the southern portion of the 450-mile-long Great Valley Geomorphic Valley (Bartow 1991). The topography of the Great Valley (commonly referred to as the San Joaquin Valley [SJV]) is relatively flat; it is bordered by the Pacific Coast Range to the west, the Klamath Mountains and Cascade Range to the north, the Sierra Nevada to the east, and San Emigdio and Tehachapi mountains to the south.

Superimposed upon this large-scale, relatively flat topography is a localized topography caused by recent incisions of river systems. This localized topography is comprised of short, steep river/stream banks with channels at lower elevations relative to the surrounding areas. These channel bottoms range between wide, relatively flat-bottomed (with occasional rounded natural levees) or narrow gully-type valleys, depending on their age and the amount of flow; however, along the CP1 alignment these features appear to have been either channelized or redirected along more convenient routes to accommodate the present urbanization.

The natural topography along the CP1 alignment is generally flat and varies between 285 and 295 feet (1988 North American Vertical Datum, NAVD88). The elevations of exploratory holes performed during the 30% Geotechnical Investigation varied between 283 and 306 feet (NAVD88). Localized variations in the ground surface elevation generally occur at existing road embankments, detention basins, and other man-made features such as irrigation canals and road crossings.

4.2 Geologic Setting

4.2.1 Regional Geology

In his discussion of geology in the southern SJV, Bartow (1991) writes that the SJV is an "asymmetric structural trough that is filled with prism sediments up to 30,000 feet thick. It formed the southern part of an extensive fore-arc basin that evolved during the Cenozoic into today's hybrid intermontane basin".

Bartow continues discussing the sedimentation infill of the SJV, stating that it

> evolved through the gradual restriction of the marine basin due to uplift and emergence of the northern Great Valley in the late Paleogene, the closing off of the western outlets in the Neogene, and finally the sedimentary infilling in the Neogene and Quaternary. These sediments rest on crystalline basement rocks of the southwestward-tilted Sierran block (1991).

4.2.2 Local Geology

According to the City of Kerman General Plan (2007), the local geology of the Kerman and Fresno area

> is created by the low alluvial fans of the perennial San Joaquin River and four ephemeral streams that form the Alluvial Fan sequence. The Pleistocene formations that make up the Fresno fan sequence are the Modesto, Riverbank, and Turlock formations.

In this report, the Modesto, Riverbank, and Turlock formations have been identified as Qf, Qc, and Qp, respectively. These deposits make up the major surface and subsurface units and originate from stream

channels emanating from the foothills east of Fresno. They are similar in mineralogy, deposition, and source.

The Modesto foundation is often referred to as a terrace deposit and

> occupies the highest stratigraphic position. Sediments within the Modesto formation range in grain size from clay to gravel and seldom exhibit well-developed sedimentary structures. The Riverbank formation underlies the Modesto formation, but does not differ greatly in lithology or texture. It is also characterized by the occurrence of a laterally extensive, but not pervasive, caliché hardpan member (Cehrs et al 1980).

Cehrs et al continue, "The Turlock formation is the oldest unit exposed in the Fresno alluvial fan sequence and forms extensive subsurface deposits throughout the SJV. It contains the majority of the hydrologically important subsurface deposits in the Fresno area" (1980).

Because of the depth of the Turlock formation, it is unlikely this unit will be encountered during construction.

South of E North Avenue there is a possibility of encountering sand dunes overlying the Modesto Formation. Aeolian sand dunes appear on some geologic maps but not others. The sand dunes have been described to have a relief of about 5 to 20 feet and are associated with a group of surface expressions that trend southeast. These dune deposits are well sorted and moderately permeable.

Additional information, including geologic cross sections, local geologic map, and hydrogeologic cross sections are presented in the GDR.

4.3 Seismic Setting

According to Jennings, the Fresno area is located "within a relatively seismically quiescent region between two areas of documented tectonic activity, the Coast Ranges-Sierran Block boundary zone to the east and the Pacific Coast Ranges boundary zone to the west" (1994).

Jennings colleagues, Unruh and Moores continue,

> The Coast Ranges-Sierran Block, which follows the physiographic boundary between the Coast Ranges and Great Valley geomorphic provinces, contains potentially active blind thrust faults. Based on the size of historical events and on the inferred subsection of the boundary zone, these blind thrust faults are capable of producing moderate to large earthquakes. (1992)

Jennings identifies the predominant source of seismic shaking in the SJV as the Pacific Coast Ranges, which contain "many active faults that are associated with the northwest-trending San Andreas Fault System" (1994). The San Andreas Fault System is the principal tectonic element of the North American-Pacific plate boundary in California.

4.3.1 Faults & Seismicity

There are no known active faults crossing or within close proximity to the alignment within the study area. Consequently, there are also no restrictions to development in the way of Alquist-Priolo earthquake fault zones as defined under the as defined by the California Division of Mines and Geology. The San Andreas Fault, located approximately 65 miles from the site, has the highest slip rate and is the most seismically active of any fault near the F-B alignment.

Caltrans used the USGS fault database to develop seismic hazard contours in 2007. The Caltrans map and Caltrans fault database were used to develop Table 4.3-1, which lists the faults within 100 miles of the

F-B CP1 alignment. Figure 4.3-1 shows the F-B CP1 alignment located approximately on the Caltrans fault map with fault identification numbers.

Table 4.3-1
Caltrans Faults, Fault Characteristics, and Distance within 100 Miles of F-B CP1

Fault Name	Caltrans Fault ID	Fault Mechanism	Maximum Moment Magnitude (Mmax)	Approximate Distance from F-B CP1 (miles)
Great Valley fault 7 through 14	25, 26, 32, 33, 34, 35, 36, 37	R	6.3-6.7	42-91
O'Neill fault	51	R	6.7	48
Ortigalita fault zone (Piedra Azul section) (Los Banos section) (Ortigalita-Cottonwood Arm section)	 389 387 388	RLSS	7.1	 61 69 67
San Andreas fault zone (Creeping section)	311	RLSS	7.9	65
Melones fault zone	50	N	6.5	86
Bear Mountains fault zone (Bowie Flat fault section) (Green Springs Run fault) (Negro Jack fault section)	 251 198 159	N	6.5	86
Kern Gorge	359	N	6.6	98
Southern Sierra Nevada fault zone (Independence section)	185	N	7.1	79
Owens Valley fault zone (1872 Rupture section)	391	RLSS	7.6	88
White Mountains fault zone (Inyo section)	419	RLSS	7.4	91
Round Valley fault	174	N	7	73
Hilton Creek fault	173	N	6.7	73
Hartley Springs fault zone	172	N	6.6	78
Silver Lake fault	171	N	6.9	78
Rinconada fault zone (San Marcos section)	400	RLSS	7.5	89
Calaveras fault zone (Southern Calaveras section) (Paicines section)	 323 324	RLSS	7.4	80
Sargent fault (Southeastern section)	405	RLSS	6.8	96

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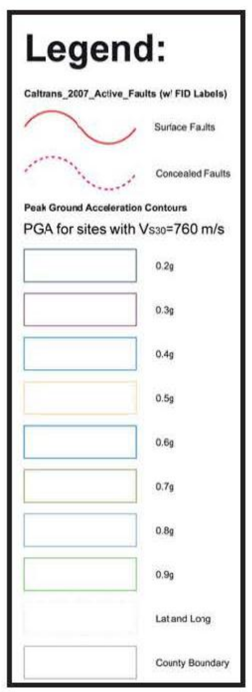
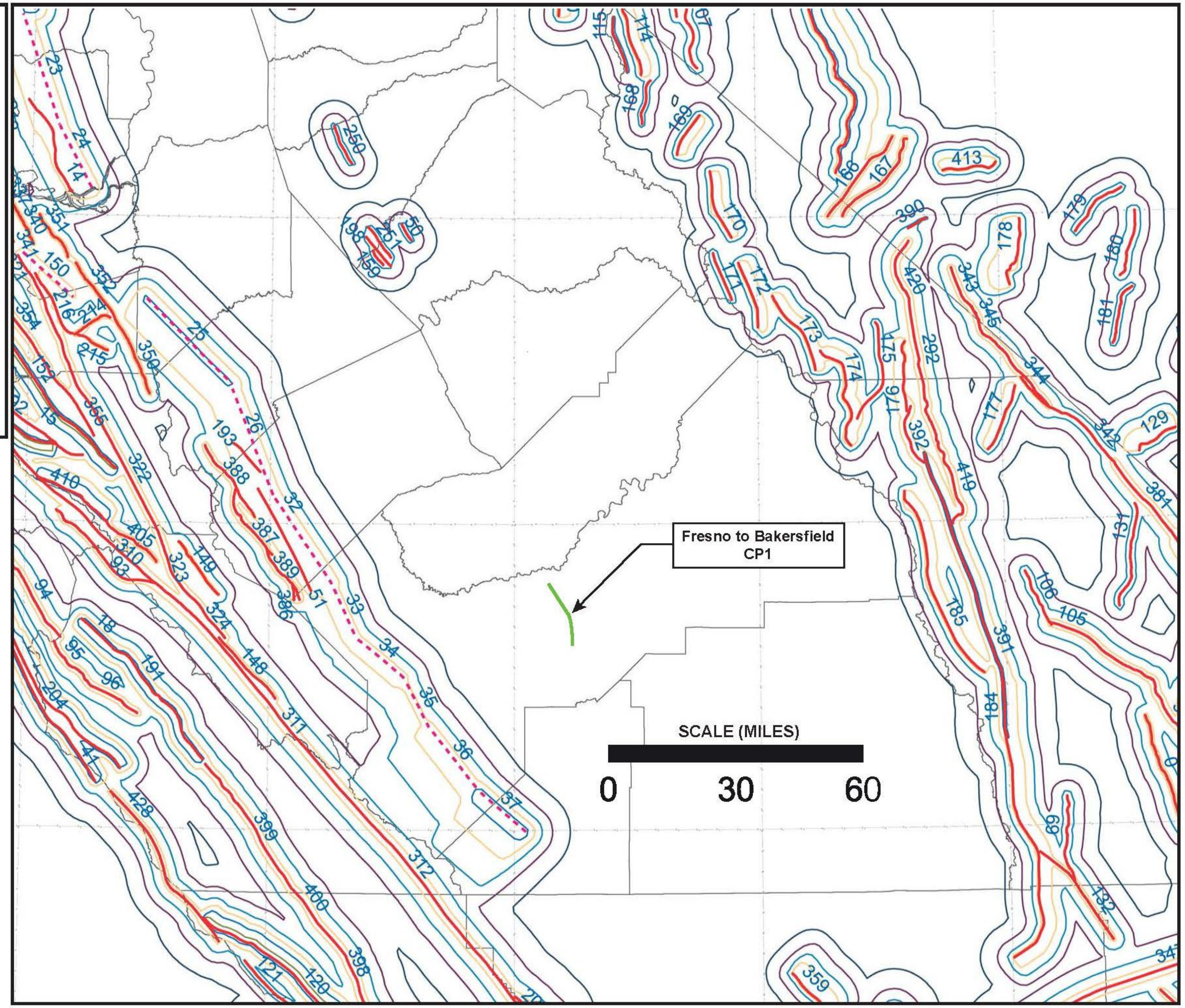
FAULT ID - FAULT NAME (FAULT MECHANISM, Mmax)
25 - Great Valley fault 7 (R, Mmax = 6.7)
26 - Great Valley fault 8 (R, Mmax = 6.6)
32 - Great Valley fault 9 (R, Mmax = 6.6)
33 - Great Valley fault 10 (R, Mmax = 6.4)
34 - Great Valley fault 11 (R, Mmax = 6.4)
35 - Great Valley fault 12 (R, Mmax = 6.3)
36 - Great Valley fault 13 (R, Mmax = 6.5)
37 - Great Valley fault 14 (R, Mmax = 6.4)
51 - O'Neill fault (R, Mmax = 6.7)
389 - Ortigalita fault zone (Piedra Azul section) (RLSS, Mmax = 7.1)
387 - Ortigalita fault zone (Los Banos section) (RLSS, Mmax = 7.1)
388 - Ortigalita fault zone (Ortigalita-Cottonwood Arm section) (RLSS, Mmax = 7.1)
311 - San Andreas fault zone (Creeping section) (RLSS, Mmax = 7.9)
50 - Melones fault zone (N, Mmax = 6.5)
251 - Bear Mountains fault zone (Bowie Flat fault section) (N, Mmax = 6.5)
198 - Bear Mountains fault zone (Green Springs Run fault) (N, Mmax = 6.5)
159 - Bear Mountains fault zone (Negro Jack fault section) (N, Mmax = 6.5)
359 - Kern Gorge (N, Mmax = 6.6)
185 - Southern Sierra Nevada fault zone (Independence section) (N, Mmax = 7.1)
391 - Owens Valley fault zone (1872 Rupture section) (RLSS, Mmax = 7.6)
419 - White Mountains fault zone (Inyo section) (RLSS, Mmax = 7.4)
174 - Round Valley fault (N, Mmax = 7)
173 - Hilton Creek fault (N, Mmax = 6.7)
172 - Hartley Springs fault zone (N, Mmax = 6.6)
171 - Silver Lake fault (N, Mmax = 6.9)
400 - Rinconada fault zone (San Marcos section) (RLSS, Mmax = 7.5)
323 - Calaveras fault zone (Southern Calaveras section) (RLSS, Mmax = 7.4)
324 - Calaveras fault zone (Paicines section) (RLSS, Mmax = 7.4)
405 - Sargent fault (Southeastern section) (RLSS, Mmax = 6.8)





Note: PGA contours shown are not for use in design or baseline.

Figure 4.3-1
Fault and Proximity to F-B CP1

4.3.2 Design Earthquake and Design Ground Motion

For the CP1 alignment, two design level earthquakes have been defined for final design per the Design Criteria Manual:

Maximum Considered Earthquake (MCE): ground motions corresponding to greater of (1) a probabilistic spectrum based upon a 10% probability of exceedance in 100 years (i.e., a return period of 950 years) and (2) a deterministic spectrum based upon the largest median response resulting from the maximum rupture (corresponding to maximum moment magnitude [M_w]) of any fault in the vicinity of the structure.

Operating Basis Earthquake (OBE): ground motions corresponding to a probabilistic spectrum based upon an 86% probability of exceedance in 100 years (i.e., a return period of 50 years).

Site-specific spectrally matched response spectra and peak ground accelerations for the Central Valley alignment between Merced and Bakersfield were developed for PEP. Peak ground accelerations and moment magnitudes used for preliminary liquefaction evaluations discussed in Section 4.3.3 are shown on Table 4.3-2. Acceleration response spectra are provided in the Design Criteria Manual.

Table 4.3-2
30% Design Seismic Parameters

Seismic Parameter	OBE	MCE
Peak ground acceleration (g)	0.08	0.25
Moment magnitude (M_w)	6.6 – 8.0	6.6 – 8.0

For each geological site and over river crossings, creeks, channels where highly compressible and loose soils may be present, site-specific subsurface data including shear wave velocity, groundwater table, soil consistency shall be obtained by the contractor and submitted to the Authority for updating final ground motion analyses, the results of which will be provided to the contractor for final design.

4.3.3 Liquefaction

Liquefaction assessments for the CP1 alignment were performed for both the OBE and MCE events using the subsurface data presented in the GDR. Based on an assumed groundwater level at 40 feet BGL, it has been concluded that soil liquefaction on a global basis is unlikely to occur following a strong earthquake on one of the nearby faults; however, localized liquefaction in discrete layers is possible.

For bidding purposes, assume liquefaction will not occur; however, the Contractor is required to perform an independent liquefaction hazard analyses for final design.

4.4 Hydrologic Setting

4.4.1 Regional

The HST alignment is located within the Kings Sub Basin. A hydrogeologic cross-section of the basin is included in the GDR. Groundwater within the northeastern quadrant of the basin is managed under the Fresno Regional Area Groundwater Management Plan. Groundwater is the sole source of drinking water in the region. The current and potential uses of groundwater in the basin are municipal and domestic supply, industrial process supply, industrial service water supply, and agricultural water supply.

Prior to urbanization and agricultural pumping, the groundwater table was within 20 or 30 feet of the ground surface. Urbanization and the pumping demand on the groundwater table have caused a cone of depression within the city of Fresno.

Groundwater well measurements by the City of Fresno indicate the groundwater table has experienced a depletion of about 60 feet since 1960 and about 100 feet since 1930. Historical groundwater levels are discussed in the GDR.

The regional groundwater flow direction in this area is from east to west. There are some localized influences as a result of both pumping, surface water treatment and groundwater recharge appurtenances.

4.4.2 Major Aquifers

The depositional environment has formed a sequence of aquifers and aquitards that vary in thickness and lateral continuity. Aquifers are generally composed of granular water-bearing sediments and aquitards are composed of finer-grained sediments that retard water flow. Three aquitards A, B, and C have been reported to exist in the vicinity of the project (CH2M Hill 2005. Most of the aquifers underlying the study area are unconfined but can be semi-confined in isolated locations. The primary aquifer in the study area is Fresno Sole Source Aquifer.

Generally, there are no extensive, low-permeability soils that isolate the upper aquifers from the lower aquifers. The Corcoran Clay (E-Clay) and correlative layers underlie the city of Fresno at a depth of about 300 feet BGL (Brown & Caldwell 2006).

4.4.3 Current Groundwater Conditions

Groundwater levels were monitoring as part of the 30% Geotechnical Investigation. Currently, the groundwater level is at about 90 feet BGL throughout the Fresno city limits and gently rises to about 60 feet BGL toward the southern reaches of the CP1 alignment. Perched groundwater was encountered during the investigation and can be encountered during construction.

Baseline groundwater levels are presented in Section 6.

4.4.4 Land Subsidence

Many areas within the SJV have experienced significant subsidence due to groundwater extraction. However, there is no documented historic land subsidence within the study area. The area may have experienced land subsidence in the early 1930s when it was prevalent in the SJV; however, no significant land subsidence is known to have occurred in the last 50 years as a result of land development, water resources development, groundwater pumping, or oil drilling.

The GDR includes the results of a cursory assessment of land subsidence made within the limits of CP1 relative to existing topography. The assessment confirmed that there has been no detectible land subsidence in the Fresno Area.

5.0 Related Construction

The following is a brief description of several large, transportation related infrastructure improvements in the vicinity of the proposed CP1 alignment. Three freeways of the California State Highway System either traverse or are adjacent to the proposed alignment, including SR 99, SR 41, and SR 180.

SR 99 in Fresno runs parallel to the proposed alignment and is generally located 0 to 0.6 miles to the west of the CP1 alignment. SR 99 was upgraded to a six-lane freeway in the 1950s with construction lasting from 1947 to 1960. The historic US-99 route followed Golden State Boulevard along surface streets through the City of Fresno. The six-lane freeway structure completed in 1960 bypassed Golden State Boulevard and is now also called the Golden State Highway. The section of the current SR 99 south of Ventura Avenue was constructed at a later date.

The following provide project background on the construction of SR 99 through Fresno. These articles do not contain technical and engineering information:

- California Highways and Public Works, 1955 "Fresno Freeway: Will Provide Many Benefits to Through and Local Traffic," September-October, pp. 27-29.
- California Highways and Public Works, 1957. "Fresno Freeway: 11-year Study Brings $11,000,000 Bypass," November-December, pp. 22-24.
- California Highways and Public Works, 1957. "The New Look: Fresno Working on Pattern of Highways," July-August, pp. 10-13.
- California Highways and Public Works, 1960. "Fresno Freeway: Northern Extension Eliminates last Three-lane Section on Highway 99," July-August, pp. 7-9.

The SR 41 freeway structure runs north-south and was constructed in the 1980s. The SR 180 freeway structure run east-west traversing the HST alignment and was constructed between 1992 and 1995.

Geotechnical logs of test borings for several structures along these freeways were collected from Caltrans database of as-built construction records. These logs of test borings are presented in Appendix A of the GDR.

Additional information regarding construction methods, ground behavior, groundwater conditions, ground support methods, and problems during construction were not described in any of the as-built construction records obtained.

6.0 Ground Characterization

6.1 Baseline Description of Subsurface Conditions

Subsurface soils have been characterized into two separate strata: (1) Existing Fill and (2) Alluvial Fan. The Alluvial Fan stratum is assumed to include the Modesto (Qf) and Riverbank (Qc) formations and Sand Dunes (Qs). The Turlock (Qp) formation was not encountered during the 30% Geotechnical Investigation and is not anticipated to be encountered during construction.

A distinction was not made between the Modesto, Riverbank, and Sand Dunes for bidding purposes because a discernible difference between their composition and engineering properties was not identified during the investigation.

Baseline descriptions for Existing Fill and Alluvial Fan are presented in Sections 6.1.1 and 6.1.2. Baseline engineering properties for Existing Fill and Alluvial Fan are described in Sections 6.2.1 and 6.2.2, respectively. General baseline descriptions for behavioral response are presented in Section 6.3.

6.1.1 Existing Fill

Existing Fill encountered during the geotechnical investigation varied from 1 to 7 feet in thickness. The depth of Existing Fill was identified chiefly from hand augering during utility clearance prior to each point of exploration. Existing Fill consists of Silty Sand (SM), Sand with Silt (SP-SM), Sandy Silt (ML), Silt (ML), and contains varying amounts of fine gravel. Ceramic and glass debris were present in Existing Fill encountered in borehole S0001R.

Existing Fill also includes surface pavements consisting of asphalt concrete (AC), concrete, and aggregate base. Where encountered, existing AC varied from 4 to 8 inches in thickness, aggregate base from zero to 9 inches; concrete for road gutter measured about 12 inches at borehole S0004R.

No historical records describing how Existing Fill was placed and compacted were located during preliminary desk studies.

On average, boreholes and CPTs from the field exploration were spaced 1 and 1/3 miles apart, respectively. Areas with deeper Existing Fill or with Existing Fill containing debris and garbage are likely present between exploratory holes. In the Fresno Area, it is not uncommon to encounter debris of unknown origin during construction excavations.

For bidding purposes, assume Existing Fill blankets the ground surface from W Clinton Avenue to E North Avenue and is present to a depth of 5 feet BGL. South of E North Avenue, assume Existing Fill blankets the ground surface and is present to a depth of 2 feet BGL.

It should be noted that in the immediate vicinity of the SR 180 overcrossing and planned Fresno Grade Separation, Caltrans as-built construction records indicate 12 to 13 feet of fill was initially placed to raise the surface elevation from 284 feet to 296 feet (NGVD29) to support the overcrossing foundations. After these foundations were constructed, approximately 20 feet of additional fill was added to raise the grade to its existing elevation of about 317 feet (NAVD88). In total, over 30 feet of fill has been added in the area where SR 180 intersects the CP1 alignment.

The nature of drilling and sampling methods used and borehole spacing makes it difficult to quantify the maximum size of fragments in Existing Fill. For bidding purposes, assume debris up to 1 foot in greatest dimension is present in Existing Fill. In addition, assume existing asphalt and concrete pavements and aggregate base rock are 9 inches thick.

According to published maps and reports from United States Department of Agriculture (USDA), Natural Resources Conservation Service (NRCS) San Joaquin, Greenfield, and Lorena soil series have hardpan

layers that are similar in composition and thickness. They are described as varying between 12 and 48 inches below the surface and between 4 and 17 inches thick. The USDA and NRCS maps indicate the hardpan is common along the CP1 alignment between W Clinton Avenue and E Church Avenue. Although this surficial layer is described in the agricultural literature, it was not encountered when hand auguring exploration locations during the 30% Geotechnical Investigation. Therefore, it is assumed that this surficial hardpan layer was either removed during urbanization in the Fresno Area or blanketed with fill such that it is currently deeper than 5 feet.

6.1.2 Alluvial Fan

The Alluvial Fan strata (Qc, Qf, and Qs) is present beneath Existing Fill to the maximum depth explored. Alluvial Fan consists of interbedded layers of poorly graded sand and silt, with varying amounts of coarse and fine grained particles. Interlayers of this unit are classified as Sand (SP), Sand with Silt (SP-SM), Silty Sand (SM), Clay (CL), Silty Clay (CL-ML), Sandy Silt (ML), Silt with Sand (ML), and Silt (ML) according to the Unified Soil Classification System (USCS).

USCS distributions are shown in Table 6.1-1 and Figure 6.1-1. The USCS identification with the largest distribution is Silty Sand, followed by Sandy Silt, Silt with Sand, Silt, Sand with Silt, and Sand. Interlayers of the Alluvial Fan strata include normalized Soil Behavior Type (SBT_N) zones 3 through 9 as classified by CPTs (Robertson 1990). The distribution of SBT_N zones according to stratum depth interval is provided in Figure 6.1-2; each test result shown corresponds to a soil layer thickness of about 0.165 feet. As shown, SBT_N zone 6 (sands – clean sand to silty sand) is mostly frequently encountered in the Alluvial Fan strata. Figure 6.1-2 shows that SBT_N zones 6 and 7 (sands and gravelly sand to dense sand) is more pronounced above 50 feet and zones 4 and 5 (silt mixtures and clay mixtures) is more pronounced below 50 feet.

Table 6.1-1
USCS Distribution for Alluvial Fan by Percentage of Depth Explored

Borehole ID	SP	SP-SM	SM	SC	SW	ML	CL	CL-ML
S0001R	18	6	40	0	0	13	10	13
S0002R	30	2	35	0	0	27	0	6
S0003R	6	25	15	0	0	26	2	26
S0004R	8	8	16	0	0	48	1	19
S0005R	6	36	12	0	0	33	0	13
S0006R	4	25	42	0	0	29	0	0
S0007R	0	19	43	0	0	33	5	0
S0010R	6	9	38	3	0	20	19	5
S0012R	5	12	45	0	0	38	0	0
S0013AR	59	20	18	0	0	3	0	0
S0014R	6	7	51	0	0	20	8	8
S0014AR	0	6	57	0	0	21	16	0
S0015R	9	5	41	0	0	33	0	12
S0016R	3	32	40	0	0	18	6	1
S0017R	19	10	30	3	0	28	7	3
S0018R	3	21	52	0	0	17	0	7
S0019R	14	37	32	0	0	17	0	0

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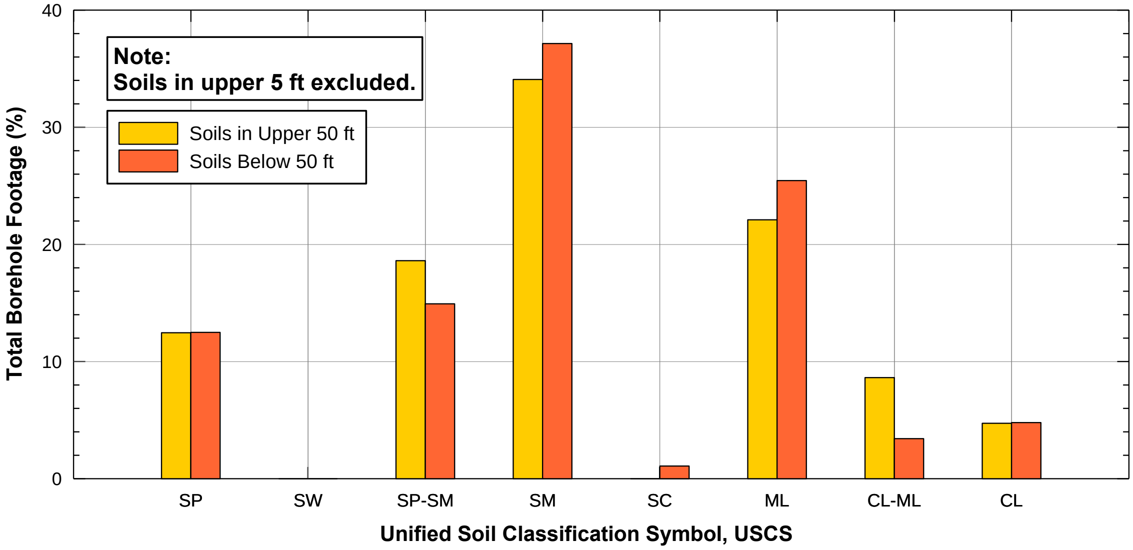

Figure 6.1-1
Unified Soil Classification System (USCS) Distribution for Alluvial Fan

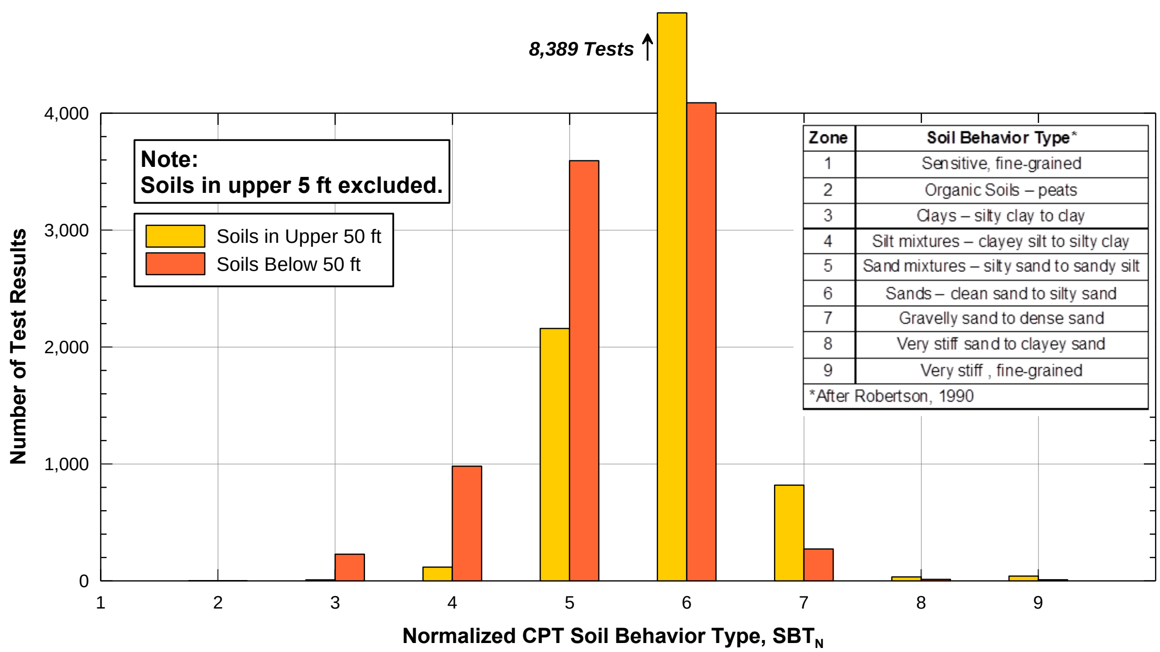

Figure 6.1-2
Normalized CPT Soil Behavior Type (SBT_N) Distribution for Alluvial Fan

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Grain size distribution for Alluvial Fan is presented in Figure 6.1-3. These curves are based on the results of laboratory sieve and hydrometer analyses performed on samples from boreholes drilled during the 30% Geotechnical Investigation. The frequency of gradation tests with depth are shown in Figure 6.1-4. Figures 6.1-3 shows that Alluvial Fan is a mixture of fine and coarse-grained soil with varying amounts of clay and few (5 to 10 percent) to trace (less than 5 percent) gravel.

For bidding purposes, assume 50 percent of Alluvial Fan encountered during construction consists of fine grained soil (finer than 0.075 mm sieve) and 50 percent consists of coarse-grained soil (coarser than 0.075 mm sieve).

According to data from the 30% Geotechnical Investigation, the coarse-grained soil is poorly graded and contains between 5 and 45 percent fine-grained soil and between zero and 5 percent fine gravel by weight. The fine-grained soil contains between 2 and 44 percent coarse grained soil. Hydrometer tests on fine-grained soils indicated clay content ranging from zero to 34 percent (percent finer than 0.002 mm) and silt content ranging from 28 to 87 percent (percent finer than 0.075 but coarser than 0.002 mm).

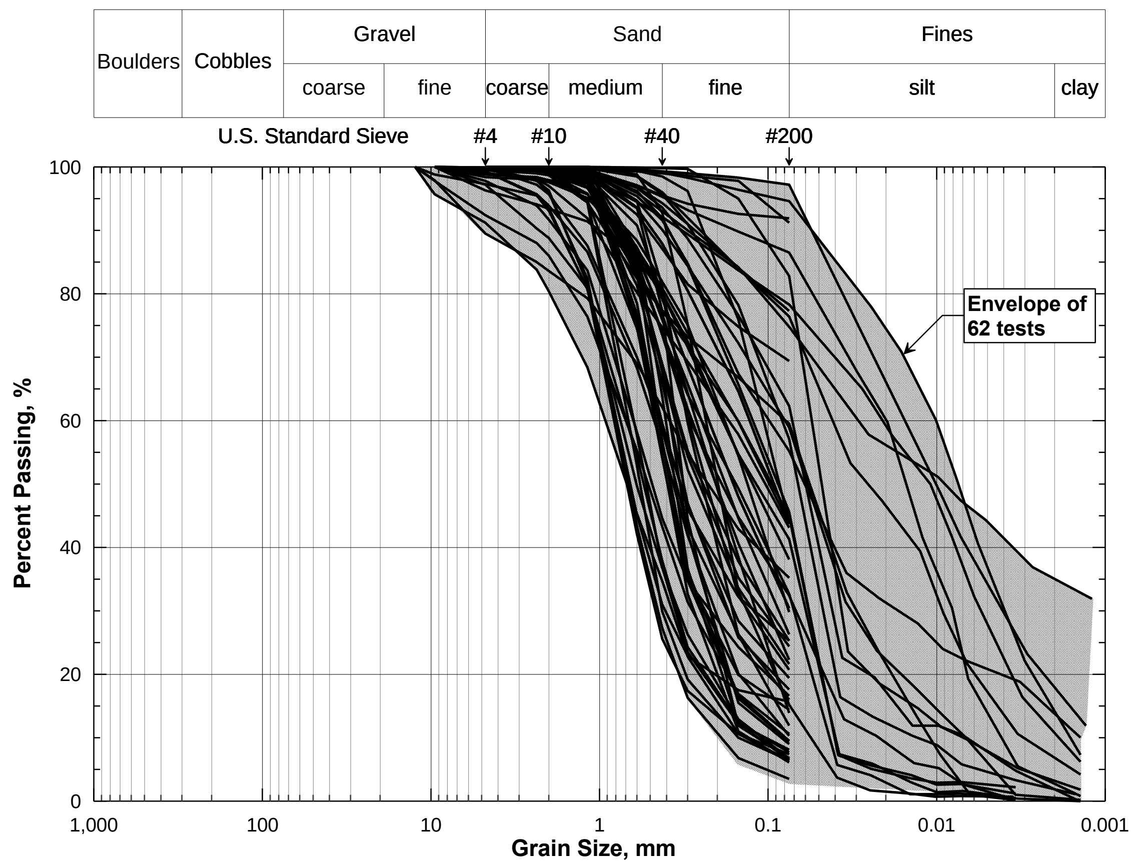

Figure 6.1-3
Representative Grain Size Distribution of Alluvial Fan

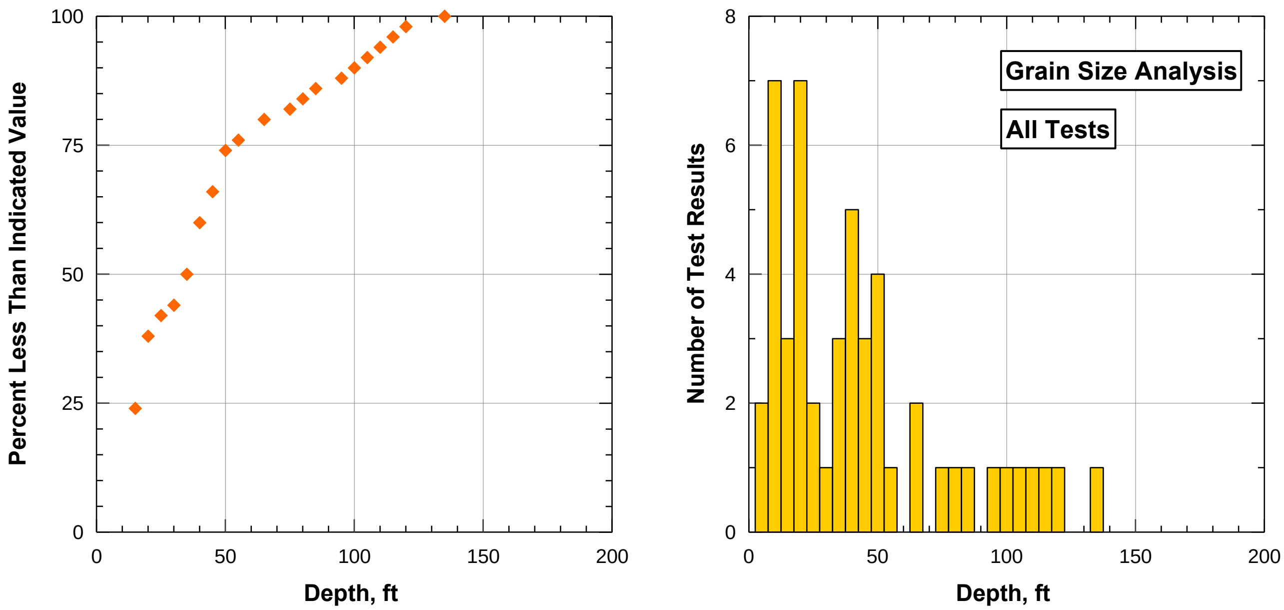

Figure 6.1-4
Probability Distribution and Frequency of Grain Size Analysis with Depth

Atterberg Limits tests were carried out on 71 samples. The results of one test indicated the soil samples were non-plastic (plastic limit could not be determined). The results of the remaining 70 tests are plotted on the Casagrande Plasticity Chart shown in Figure 6.1-5. The frequency of Atterberg Limits tests with depth is shown in Figure 6.1-6. All fine-grained soils tested were inorganic and plotted within the USCS identifications of CL, CL-ML, and ML.

The distribution of plasticity characteristics and associated USCS classification for fine grained soils within the Alluvial Fan strata are shown on Figure 6.1-5. As a baseline, coarse-grained Alluvial Fan soils should be assumed as non-plastic.

The baseline Plasticity Index for fine-grained Alluvial Fan soils is 8 and the Liquid Limit baseline is 27. These values represent the median Plasticity Index and Liquid Limit results from the available data.

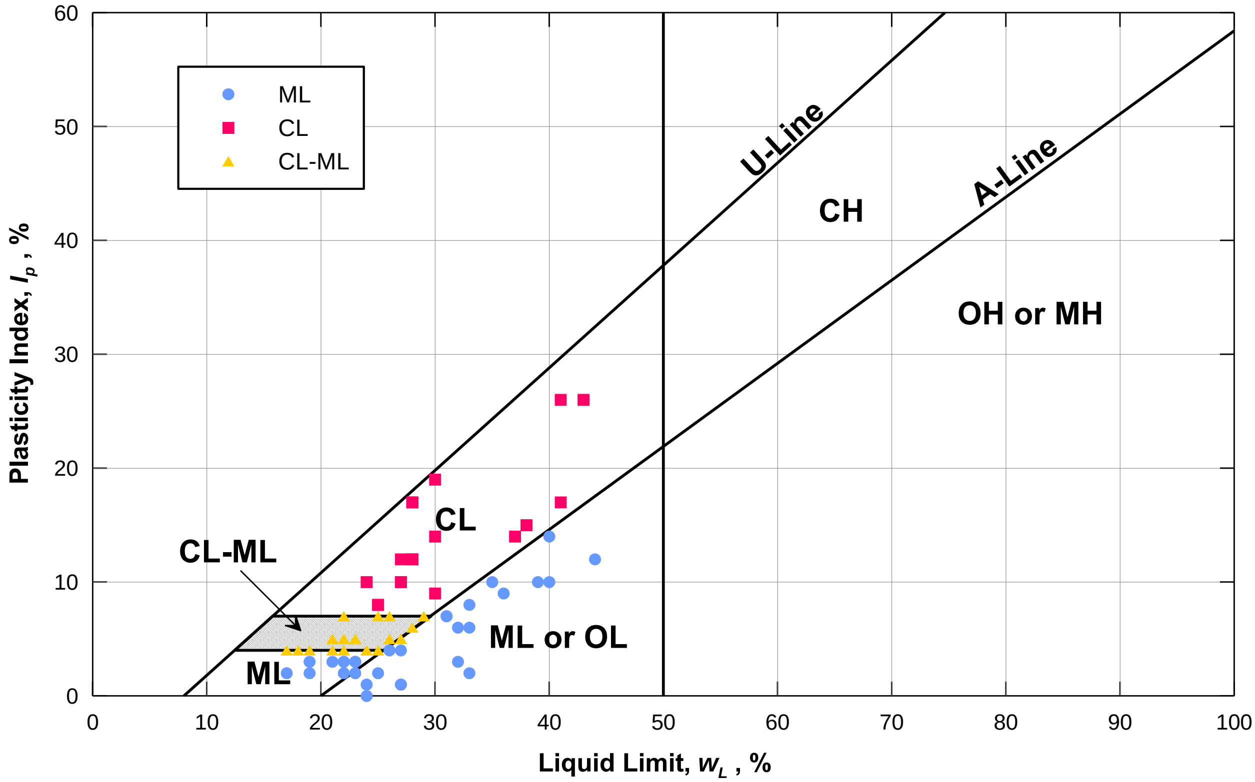

Figure 6.1-5
Representative Distribution of Plasticity Characteristics

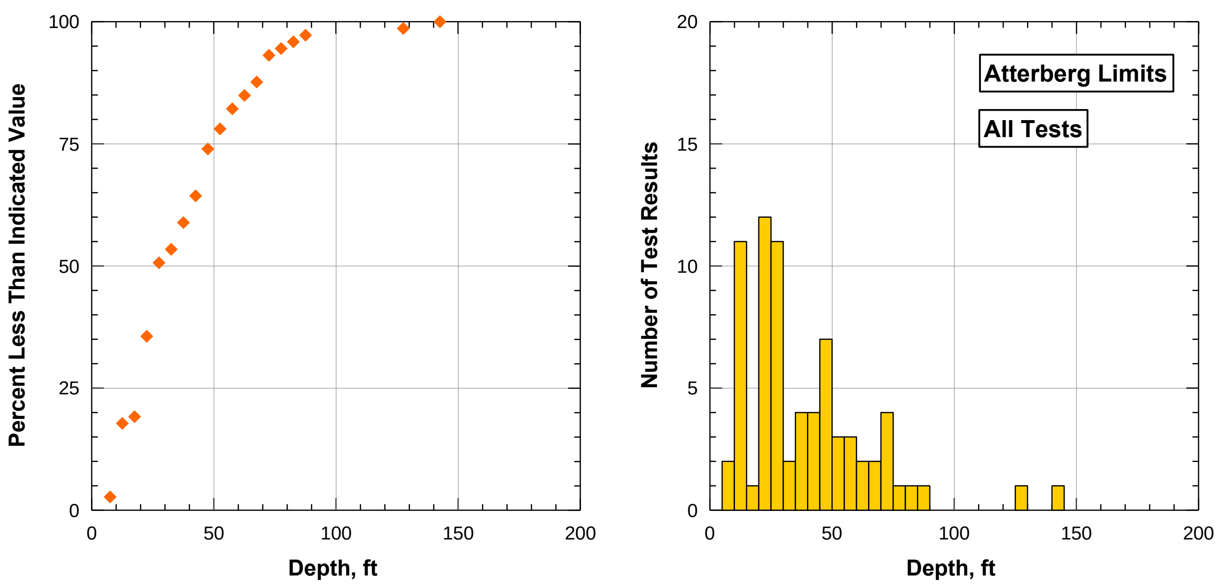

Figure 6.1-6
Probability Distribution and Frequency of Atterberg Limits Tests with Depth

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Dense, cemented soil (hardpan) is present within Alluvial Fan sequence at variable depths. In general, results from the 30% Geotechnical Investigation indicate hardpan layer varying from 1 to 12 feet in thickness are present between 5 and 50 feet BGL as evidenced by SPT blowcounts, CPT cone resistance, and pre-drilling depths for some CPTs.

Where sampled, hardpan is hard and very dense and consists of Sandy Silt, Silt, Silt with Sand, Silty Clay, Silty Sand, Sand with Silt, and Sandy Clay. SPT (N_{60}) blow count on hardpan is greater than 50 blows per foot in ML, CL, CL-ML and greater than 100 blows per foot in SM and SP-SM. Hardpan includes SBT_N zones 6 through 9 and exhibits corrected CPT cone resistance (q_t) greater than 500 tons per square foot.

Gravel is present in trace (less than 5 percent) amounts and consists primarily of granitic, metamorphic, and occasional volcanic origin. Cobbles and boulders were not encountered during our exploration.

6.1.3 Groundwater Level

In general, groundwater-levels were not measured during the 30% Geotechnical Investigation because they were obscured by the borehole drilling fluid. In the CPTs, groundwater measurements were recorded if the presence of free groundwater was suspected when CPTs were advanced or retrieved. Porewater pressure dissipation tests were performed in some CPTs to estimate groundwater levels along the alignment. Groundwater measurements and results of pore pressure dissipation tests are presented in the GDR.

Prior to urbanization and agricultural pumping, the groundwater table was within 20 to 30 feet BGL. Since about the 1960s, the groundwater table has experienced a depletion of about 50 feet. Current groundwater table measurements from standpipe piezometers and CPT pore pressure dissipation tests indicate the groundwater table is below 80 feet BGL in boreholes S0003R and S0005R and between 70 and 99 feet in boreholes S0010R, S0013AR, S0016R, S0017R, and S0018R. Groundwater measurements from CPTs indicate it is between 61 and 95 feet in CPTs S0023ACPT, S0035CPT, S0036CPT, and S0041CPT. Regional groundwater levels from the California Department of Water Resources indicate the depth to groundwater varies from about 65 to 110 feet along the project alignment.

For bidding purposes, assume the groundwater table during construction is at a depth of 90 feet BGL between W Clinton and E Church Avenues and 60 feet between E Church and E American Avenue with an allowance of plus-or-minus 5 feet for seasonal fluctuations. These numbers are based on the minimum depths measured in these sections of the alignment.

For design of permanent structures, assume a baseline groundwater table depth of 40 feet BGL during the project design life.

6.1.4 Contaminated Soil

Current and historical land use in the vicinity indicates man-made hazardous materials are likely to exist throughout the areas in and around the CP1 alignment. Hazardous materials associated with man-made contamination can include petroleum hydrocarbons, volatile organic compounds, semi-volatile organic compounds, pesticides, PCBs, and metals. These contaminants are usually associated with former agricultural, industrial, and/or commercial land uses. Aerially deposited lead is common in soil along shoulders of major thoroughfares from past leaded fuel vehicle emissions.

Railroads have historically used lead arsenate or other arsenic compounds as pesticide and herbicide, as well as using chlorinated pesticides. Lead may also be present as lead-based paint debris, or as aerially-deposited lead. PCBs were historically used in railroad electrical equipment likely up until the time they were banned in 1979. The ground investigation did not include sampling or analysis for PCBs.

Evidence of contamination was noted by strong hydrocarbon odors in two samples taken from borehole S0012R. The contaminated SPT samples were taken at depths of 25 and 30 feet BGL. A strong hydrocarbon odor was also noted in the upper 20 feet from CPT S0019CPT and a pinkish-red contaminant was observed in a soil sample from S0014AR taken between 11 and 12.5 feet. No other evidence of soil contamination was noted in any of the other samples collected during the investigation. However, uninvestigated contaminated soil could exist at other locations along the alignment.

Performing a Phase I or Phase II Environmental Site Assessment or performing analytical testing on soil and groundwater samples was beyond the scope of the investigation. Refer to the project EIR/EIS for programmatic evaluation of the potential for hazardous materials contamination of the soils.

The greatest percentage of soil to be excavated is from construction of the Fresno Grade Separation, Jensen Trench, and foundations for the proposed Fresno Viaduct and vehicular overcrossings. A smaller percentage of excavation will be from earthwork for at-grade alignment and for construction of underground utilities. For bidding purposes, assume 5 percent of all excavated soil will be considered hazardous waste and will require disposal at a Class I facility and 10 percent of all excavated soil will require disposal at a Class II facility. The remaining soil excavated is anticipated to be reused on site as Structural Fill, Embankment Fill, or backfill along other portions of HST alignment provided the gradation requirements described in the Contract Documents are satisfied.

6.1.5 Corrosive Soil

Corrosion tests were performed on 37 representative soil samples to evaluate the corrosion potential for buried iron, steel, mortar-coated steel, and reinforced concrete structures. Baseline values of soil corrosion parameters for Existing Fill and Alluvial Fan are presented in Table 6.1-2.

Table 6.1-2
Baseline Corrosion Parameters

Test	Test Reference	No. of Tests	Range of Values	Mean Value	Standard Deviation	Assumed Baseline
Minimum Resistivity (ohm-cm)	ASTM G 57	37	1,130 to 20,900	6,526	4,457	**2,000**
pH	ASTM D 4327	37	6.9 to 8.4	7.6	0.3	**6.9**
Chloride (ppm)	ASTM D 4327	37	6.2 to 124.0	15.0	20.5	**124**
Sulfate (ppm)	ASTM D4327	37	0.8 to 273.1	28.8	47.5	**273**

6.1.6 Groundwater Chemistry

Groundwater quality parameters are based on the results of 3 samples collected from the existing ground water monitoring wells. The mean value of parameters tested represents baseline condition and are shown on Table 6.1-3.

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Table 6.1-3
Baseline Groundwater Chemistry Parameters

Test	Test Reference	Borehole ID			Assumed Baseline (mean)
		S0016R	S0017R	S0018R	
pH	SM 4500-H^+B	7.51	7.24	7.51	**7.4**
Calcium (mg/L)	EPA 200.7	88	78	47	**70**
Bicarbonate Alkalinity as $CaCO_3$ (mg/L)	SM 2320B	280	260	220	**250**
Specific Conductance (umhos/cm)	SM 2510B	1,100	860	570	**840**
Total Dissolved Solids (mg/L)	SM 2320B	740	580	380	**570**
Chloride (mg/L)	EPA 300.0	83	49	23	**50**
Sulfate as SO_4 (mg/L)	EPA 300.0	53	110	21	**61**

6.2 Engineering Properties of the Subsurface Materials

6.2.1 Existing Fill

Few laboratory tests were performed on Existing Fill because the bulk samples collected were highly disturbed and were taken from drilling cuttings. Laboratory tests performed included Modified Proctor Compaction, California Bearing Ratio (CBR), moisture content, and fines content. These tests were performed to evaluate pavement design and earthwork considerations.

Baseline engineering properties of the Existing Fill are described in Table 6.2-1. The mean value from the Modified Proctor Tests was selected as the baseline for Maximum Dry Density and Optimum Moisture Content. However, the baseline values cannot be assumed without verification of similar soil conditions during earthwork inspection services. For pavement design, the minimum California Bearing Ratio test result is assumed as the baseline value. The CBR baseline represents a pavement subgrade condition that has been prepared in accordance with the Contract Documents.

Laboratory and in situ tests were not performed to measure unit weight or strength; therefore, these assumed baseline parameters are based on previous experience and engineering judgment.

Table 6.2-1
Baseline Engineering Properties for Existing Fill

	Depth	Total Unit Weight (γ_t)	Dry Unit Weight (γ_d)	Water Content (w_c)	Fines Content	Maximum Dry Density ($\gamma_{d,max}$)	Optimum Moisture Content (w_o)	California Bearing Ratio	Effective Friction Angle (Φ')
	(ft)	(pcf)	(pcf)	(%)	(%)	(pcf)	(%)		(deg)
No. of Tests		-- *	-- *	2	17	9	9	9	-- *
Range	--	104-143	100-125	4-15	13-68	121-137	6-12	13-50	-- *
Assumed Baseline	**0 to 5**	**120**	**112**	**7**	**13-68**	**130**	**8**	**13**	**28**

Note: --* indicates laboratory tests were not performed.

Bulking/swell factors used to estimate earthwork volumes typically range between 10 percent for sand and gravel to about 30 percent for clay. Shrinkage factors range from about 10 percent for sand to about 30 percent for clay. For bidding purposes, assume Existing Fill has a bulking/swell factor of 20 percent and a shrinkage factor of 10 percent.

6.2.2 Alluvial Fan

Baseline parameters for the Alluvial Fan are sorted by structure. The boreholes and CPTs contributing to the statistical evaluation of the soils at each structure are shown on Table 6.2-2

Table 6.2-2
In Situ Tests by Structure Type

Structure Name	Boreholes	CPTs
Fresno Grade Separation and Jacked-Box Tunnel	S0002R, S0003R, S0004R, S0005R, S0006R	S0006ACPT, S0007CPT, S00008CPT, S00009CPT, S0010CPT, S0011CPT, S0012CPT, S0013CPT, S0014CPT, S0015CPT
Jensen Trench	S0014AR, S0014R, S0015R	S00027CPT, S0028CPT, S0029CPT
Fresno Viaduct	S0016R, S0017R, S0018R	S0030CPT, S0031CPT, S0032CPT, S0033CPT, S0034CPT, S0034ACPT, S0035CPT, S0036CPT
At-grade, embankment, and other ancillary structures	S0001R, S0007R, S0010R, S0012R, S0013AR, S0019R,	S0001CPT, S0002CPT, S0003CPT, S0004CPT, C0005CPT, S0006CPT, S0016CPT, S0017CPT, S0018CPT, S0019CPT, S0020CPT, S0021CPT, S0022CPT, S0023CPT, S0024CPT, S0025CPT, S0026CPT, S0037CPT, S0038CPT, S0039CPT, S0040CPT, S0041CPT, S0042CPT

The baseline engineering properties of Alluvial Fan are shown on Table 6.2-3. The range of conditions and uncertainties for the parameters in Table 6.2-3 are described in Appendix A.

Table 6.2-3
Baseline Engineering Properties for Alluvial Fan

Structure Name		Depth (ft)	Total Unit Weight, γ_t (pcf)	Water Content, w_c (%)	Soil Modulus, E_s (tsf)	Corrected Blow Count, SPT N_{60} (bpf)	CPT Tip Resistance, q_c (tsf)	Effective Friction Angle, Φ' (deg)	Effective Cohesion Intercept, c′ (psf)	Shear Wave Velocity, V_{s30} (ft/sec)	Modulus of Vertical Subgrade Reaction, k_v (tons/ft^3)
Fresno Grade Separation and Jacked Box Tunnel	Range	<50	95-137	4-33	96-1,565	5-94	34-937	20-50	7-1042	1,414-1,625	60 - 300
	Baseline		**129**	**15**	**500**	**42**	**190**	**41**	**250**	**1,500**	**250**
	Range	>50	91-137	13-36	186-505	44-95	17-998	34-50	NA	1,414-1,625	175 - 300
	Baseline		**131**	**20**	**500**	**65**	**280**	**41**	**250**	**1,500**	**300**
Jensen Trench	Range	<20	95-137	7-22	177-1,178	4-99	42-657	31-50	24-269	1,027-1,197	60 - 300
	Baseline		**122**	**10**	**409**	**24**	**170**	**37**	**30**	**1,120**	**150**
	Range	>20	91-137	8-34	161-884	13-99	14-657	35-50	NA	1,027-1,625	175 - 300
	Baseline		**133**	**15**	**511**	**58**	**200**	**42**	**120**	**1,120**	**300**
Fresno Viaduct	Range	<60	90-137	10-25	184-1,829	3-98	11-1,630	31-50	75-532	1,027-1,179	60 - 300
	Baseline		**126**	**10**	**500**	**35**	**140**	**37**	**220**	**1,090**	**225**
	Range	>60	94-137	12-34	154-653	24-99	50-978	26-50	83-954	1,027-1,179	175 - 300
	Baseline		**132**	**15**	**500**	**55**	**150**	**37**	**250**	**1,090**	**300**
At-Grade, Embankment, and Other Ancillary Structures	Range	<20	90-137	5-33	80-1,807	3-100	8-1,630	20-51	30-312	1,027-1,625	60 - 300
	Baseline		**121**	**5**	**373**	**22**	**150**	**37**	**90**	**1,300**	**150**

Maximum dry density, optimum moisture content, bulking/swell factor, and shrinkage factor previously assumed for Existing Fill are also applicable for Alluvial Fan. Grain size and plasticity baseline statements were presented in Section 6.2.1.

A. Standard Penetration Test Blow Count

The baseline SPT N_{60} blow count shown in Table 6.2-3 is selected as the median from the SPT data set for each structure.

SPT blow counts were recorded during soil sampling in boreholes and corrected to SPT N_{60} values using the results of hammer efficiency measurements recorded during the site exploration. For comparison, CPT tip resistance data was correlated to equivalent SPT N_{60} values as described in Appendix A.

Histograms and statistical data of SPT N_{60} for each structure and depth interval shown in Table 6.2-3 are presented in Appendix A. Histogram plots were capped at a maximum value of 99 blows per foot.

Hardpan soils can be identified from the N_{60} values corrected for overburden (referred to as $(N_1)_{60}$). Figure 6.2-1 shows the variation of $(N_1)_{60}$ with depth for all SPT data. The overburden was calculated using a correlation to unit weight from Robertson (2009) and the results from S0009CPT. Figure 6.2-1 shows that hardpan soils are identifiable by high blow counts predominantly in the uppermost 20 feet.

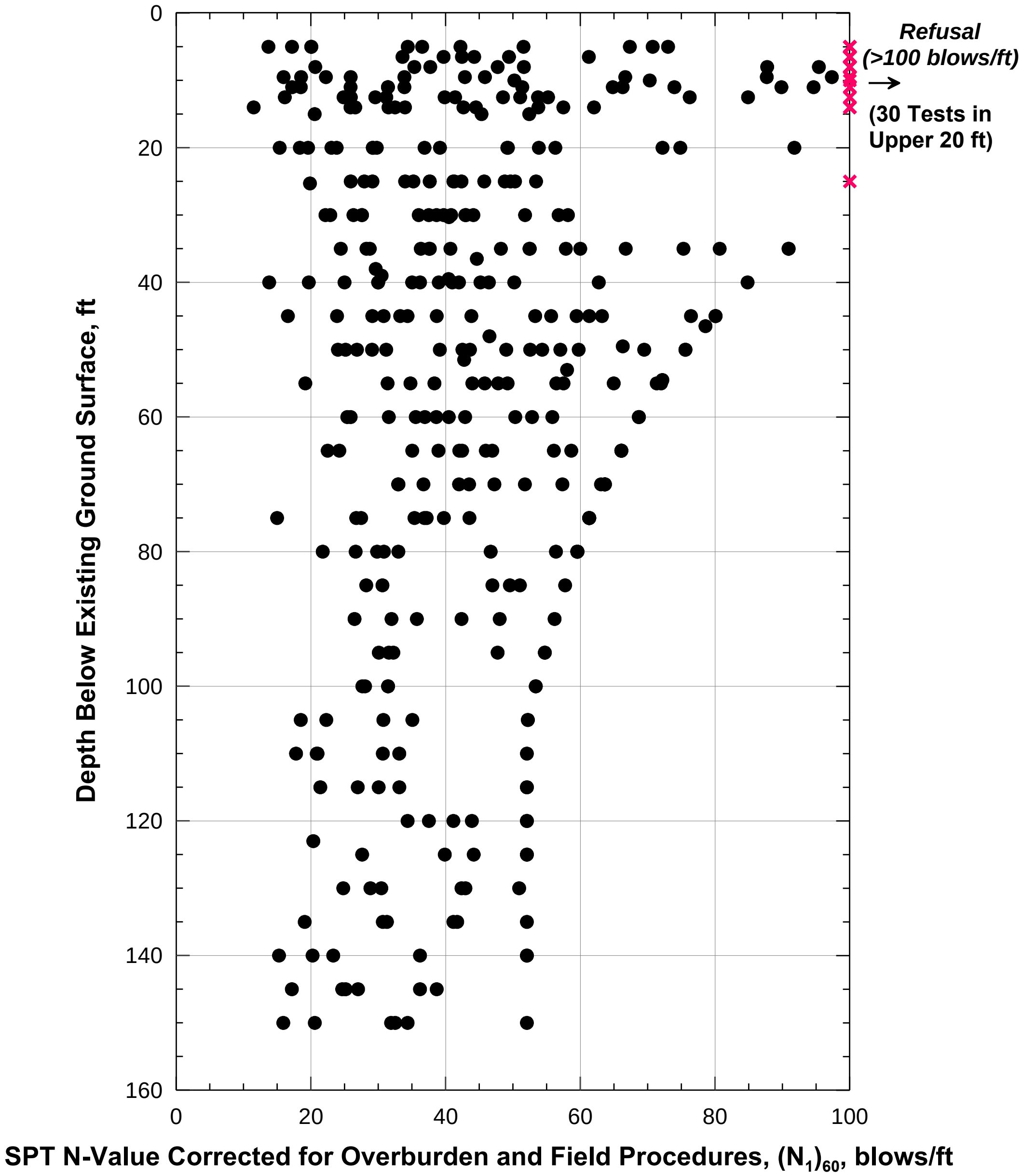

Figure 6.2-1
Results of SPT $(N_1)_{60}$ for CP1 Showing Likely Hardpan Depth

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B. Cone Penetration Test Tip Resistance

The baseline CPT tip resistance (q_c) in Table 6.2-3 is selected as the median q_c value from the CPT data set for each structure.

CPT tip resistance data for each structure, including mean, median, and standard deviation results, are presented in Appendix A.

C. Unit Weight and Moisture Content

The total unit weight baseline shown in Table 6.2-3 is selected as the median value correlated from the CPT data set for each structure.

Total unit weight (γ_t) was correlated to CPT data and plotted as histograms, as shown in Appendix A. The correlated unit weights were converted to pounds per cubic foot and capped at 137 pounds per cubic foot.

A total of 67 moisture content tests were performed on samples from boreholes S0001R through S0019R. Moisture content results ranged from 3.9 to 43.5 percent. The moisture content results from some laboratory tests could be wetter than in situ as a result of the influence from rotary wash drilling on the sample water content. Conversely, the laboratory results could be drier than in situ conditions because the SPT sampler was not equipped with liners and samples were stored in jars. Therefore, the baseline moisture content shown on Table 6.2-3 is determined on engineering judgment and our understanding of conditions at the site.

D. Shear Strength

Shear strength parameters include effective friction angle (Φ′) and effective cohesion (c′). The effective friction angle for predominantly coarse grained soil (SP, SP-SM, SM, and SC) was determined from CPT and SPT blow count correlations and, also, from the results of direct shears tests on reconstituted samples. The statistical results of the CPT and SPT correlations and laboratory test data are presented in Appendix A.

Strength parameters were estimated from the results of 45 direct shear tests on remolded soil samples collected during the field exploration. The direct shear test results from S0006R at 38 feet, S0016R at 150 feet, S0018R at 65 feet, and S0018R at 130 feet were not used in developing the strength baseline because the cohesion intercept results were unusually high and assumed to misrepresent in situ conditions.

The baseline effective cohesion shown in Table 6.2-3 is selected as the median value from remolded direct shear tests results, but is capped at 250 psf.

Figure 6.2-2 shows the stress envelopes from all direct shear tests performed for references purposes.

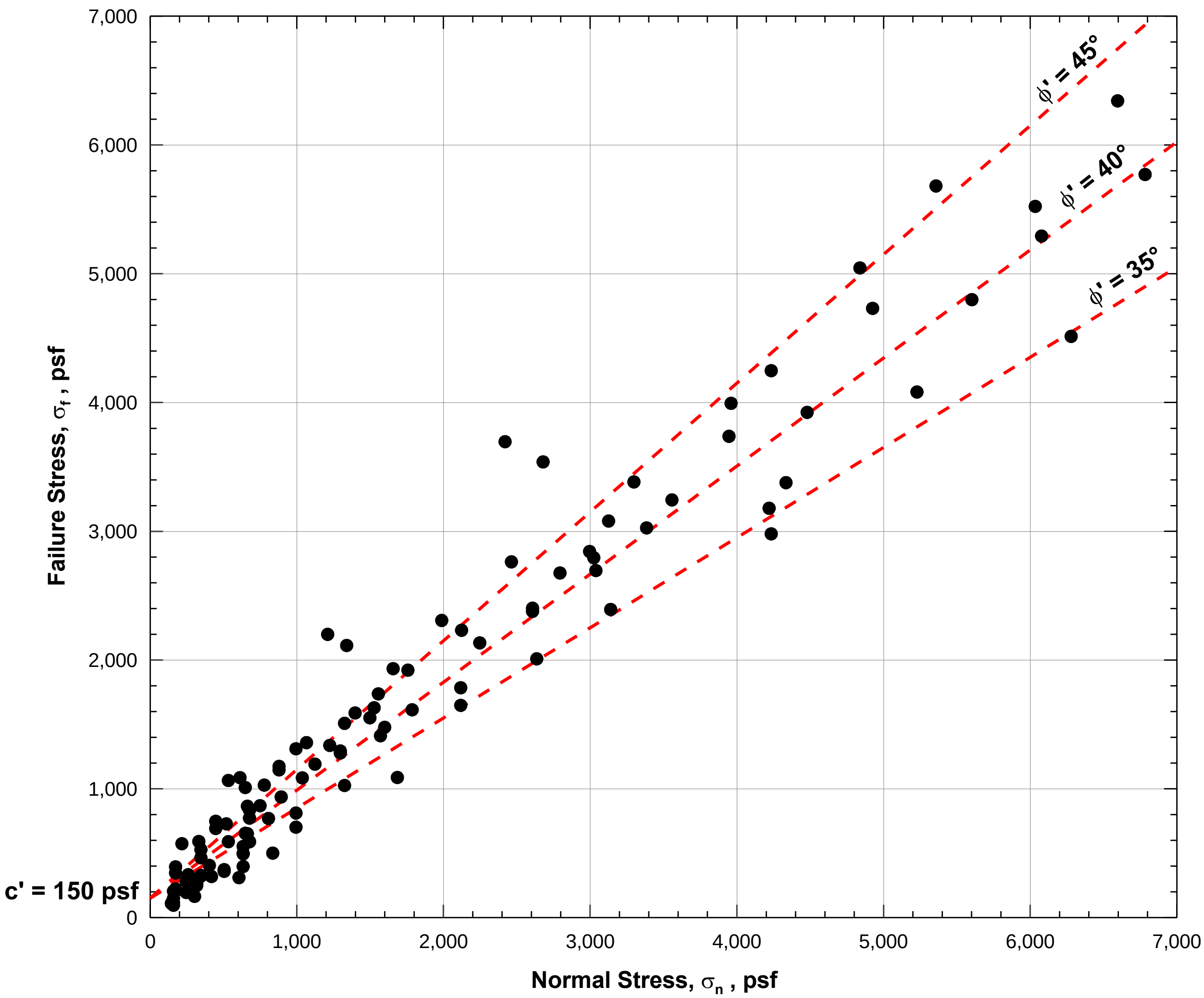

Figure 6.2-2
Results of Remolded Direct Shear Tests

E. Soil Modulus

Typical values of Soil Modulus are described in the American Association of State Highway Transportation Officials' (AASHTO) LRFD Bridge Design Specifications, Fifth Edition (AASHTO 2010) for different soil types as shown in Table 6.2-4.

The baseline Soil Modulus (E_s) shown on Table 6.2-3 is selected as the lesser of the following:

1. Median value from SPT correlation
2. Minimum value presented in Table 6.2-4 for sand with density determined from the baseline blowcount.

Histograms and other statistical data used to determine Soil Modulus from SPT and CPT correlations are presented in Appendix A.

Table 6.2-4
Published Soil Modulus (AASHTO 2010)

Soil Modulus, E_s (tsf)	
Silt	
20 to 200	
Sand	
Loose	100 to 300
Medium Dense	300 to 500
Dense	500 to 800

Figure 6.2-3 shows the estimated Soil Modulus using an SPT correlation for clean sands and slightly silty sands, published by AASHTO (2010).

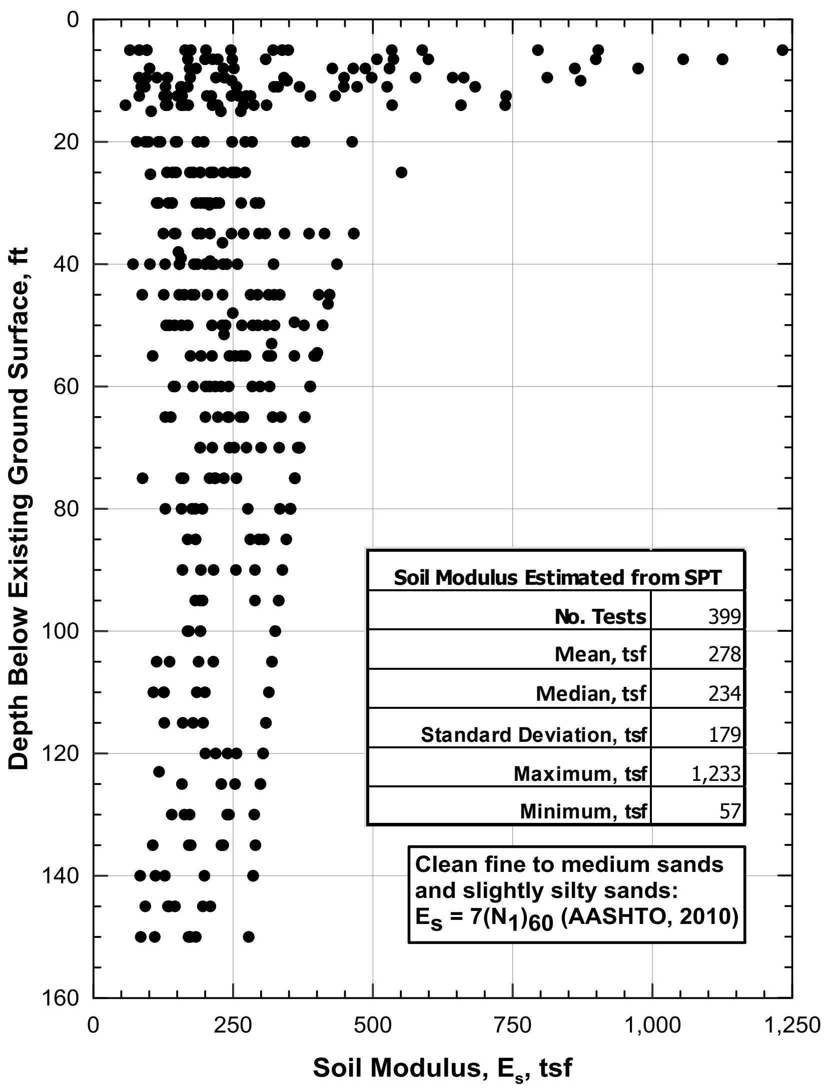

Figure 6.2-3
Soil Modulus Correlations from SPT Data

F. Shear Wave Velocity

Shear wave velocities averaged over the upper 100 feet (~30 meters) of soil, V_{s30}, are presented in the GDR. Baseline shear wave velocities are shown on Table 6.2-3.

For the Fresno Grade Separation, the baseline V_{s30} shown on Table 6.2-3 is the mean from the results of seismic CPTs and PS Logging from S009CPT, S0012CPT, and S0005R. For the Fresno Viaduct, the baseline V_{s30} is the mean from the results of seismic CPTs and PS Logging at S0030CPT, S0033CPT, and R0018R. For the Jensen Grade separation, the baseline V_{s30} is the mean from shear wave measurements south of SR41 (S0024CPT, S0030CPT, S0033CPT, and S0018R). For at-grade, embankment, and ancillary structures, the baseline value shown is the mean from all measurements recorded.

Figure 6.2-4 shows shear wave velocity profiles measured during the 30% Geotechnical Investigation. The seismic Site Class boundary between Class C and Class D soil is shown for reference only.

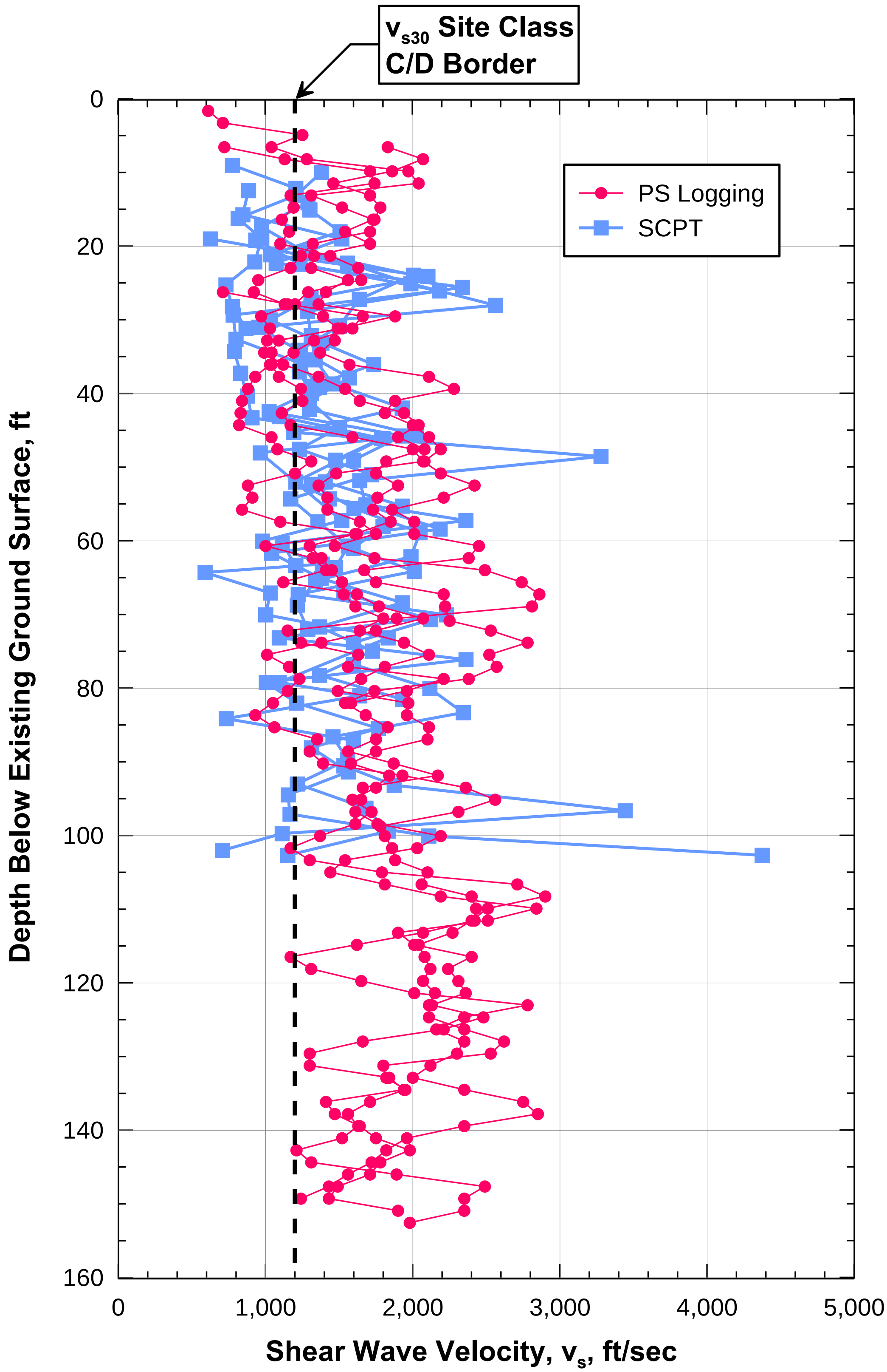

Figure 6.2-4
V_{s30} Measurements

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G. Modulus of Vertical Subgrade Reaction

Figure 6.2-4 shows the range of Modulus of Vertical Subgrade Reaction (k'_v). The baseline subgrade modulus shown on Table 6.2-3 is determined from the baseline SPT N_{60} blow count correlated to the typical vertical subgrade reaction modulus values shown in Figure 6.2-5. A bi-linear relationship between subgrade modulus and relative density was utilized.

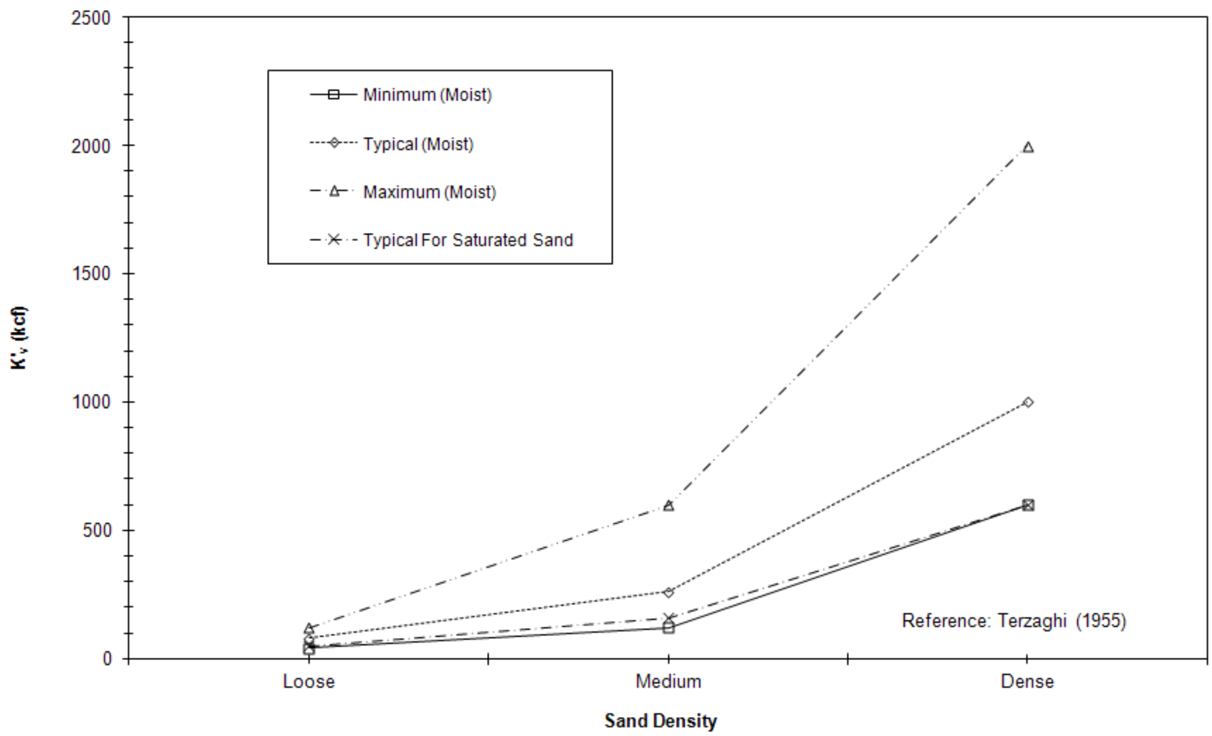

Figure 6.2-5
Modulus of Vertical Subgrade Reaction

H. Modulus of Horizontal Subgrade Reaction

Typical values of Modulus of Horizontal Subgrade Reaction (k_h) for granular soil range from 20 to 225 pounds per cubic inch based on an assessment of the relative density of the sand and the effect of a submerged or dry condition (FHWA-NHI-10-16). Typical values of k_h published by the American Petroleum Institute (API 1987) are shown on Table 6.2-5.

Table 6.2-5
Static Modulus of Horizontal Subgrade Reaction, k_h (API 1987)

	Subgrade Reaction k_h by Relative Density (pci)		
	Loose	**Medium Dense**	**Dense**
Sand Below Water Table	20	60	125
Sand Above Water Table	25	90	225

For bidding purposes, assume a modulus of horizontal subgrade reaction (k_h) of 80 pci for Alluvial Fan under static loading, and 40 pounds per cubic inch under cyclic loading.

6.3 Baseline Soil Behavior

Behavioral baselines for the preliminary design will be a function of the equipment and means and methods selected by the Contractor.

6.3.1 Existing Fill

For bidding purposes, assume Existing Fill is loose to medium dense and soft to stiff and can be excavated with conventional grading equipment such as dozers, scrapers, and track mounted excavators. Where excavated vertically, Existing Fill will not remain stable. Excavations in Existing Fill will be prone to raveling within a few minutes where it is dry, and will flow where it is wet. It is anticipated that sloped cuts or temporary shoring will be required to maintain stability of excavation in Existing Fill.

Existing Fill will require moisture conditioning prior to reuse and recompaction to achieve desired density. This will require adding water to soil that is dry of the optimum moisture content and air drying soil that is wet of the optimum moisture content. Air drying during periods of rain (November through March) is assumed to be impractical. Stabilization through lime treatment should not be considered as the fine grained soil is predominantly silty and will not have a strong reaction with lime. Cement treatment would be appropriate, but for bidding purposes assume it will not be necessary.

6.3.2 Alluvial Fan

A. Hardpan

Hardpan will be encountered during excavations for the proposed Fresno Grade Separation, Jacked Box, Jensen Trench, and during excavation for new foundations for the proposed Fresno Viaduct and other ancillary structures. Where encountered, it is difficult to excavate with conventional equipment such as track mounted excavators, and scrapers. Hardpan is also difficult to excavate with conventional solid flight augers.

Hardpan is difficult to excavate when dry but may lose its strength and become easily remolded when saturated leading to reduced bearing and lateral capacity. For this reason, hardpan within 5 feet of the ground surface should not be relied upon for support of permanent structures.

B. Cementation (Rippability)

The predominantly coarse grained soil encountered exhibits no cementation to moderate cementation according to the Soil and Rock Logging Classification and Presentation Manual (Caltrans 2010) and shown on Table 6.3-1.

Table 6.3-1
Cementation Criteria (Caltrans 2010)

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure
Moderate	Crumbles or breaks with considerable finger pressure
Strong	Will not crumble or break with finger pressure

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For bidding purposes, assume the Existing Fill and Alluvial Fan strata exhibit weak cementation.

C. Stability

For bidding purposes, Alluvial Fan above the groundwater table is assumed to be firm and to remain stable for sufficient time to allow for temporary shoring installation. Alluvial fan below the groundwater table will experience sloughing or running conditions. Therefore, where deep foundations extend below the groundwater level for construction, temporary casing and/or drilling slurry will be required.

For bidding purposes, assume Existing Fill is classified as Cal/Occupational Safety and Health Association (Cal/OSHA) Type B soil and Alluvial Fan is Classified as Type C soil.

D. Shrink/Swell Potential

Results of Atterberg limits tests indicate Alluvial Fan and Existing Fill have a low degree of shrink and swell potential associated with a mean Plasticity Index of less than 18 percent (Holtz 1959 and USBR 1974).

The shrink/swell potential can also be estimated using soil dry density correlated from CPT data and laboratory moisture content tests, and liquid limit results from laboratory tests. The results are shown in Figure 6.3-2.

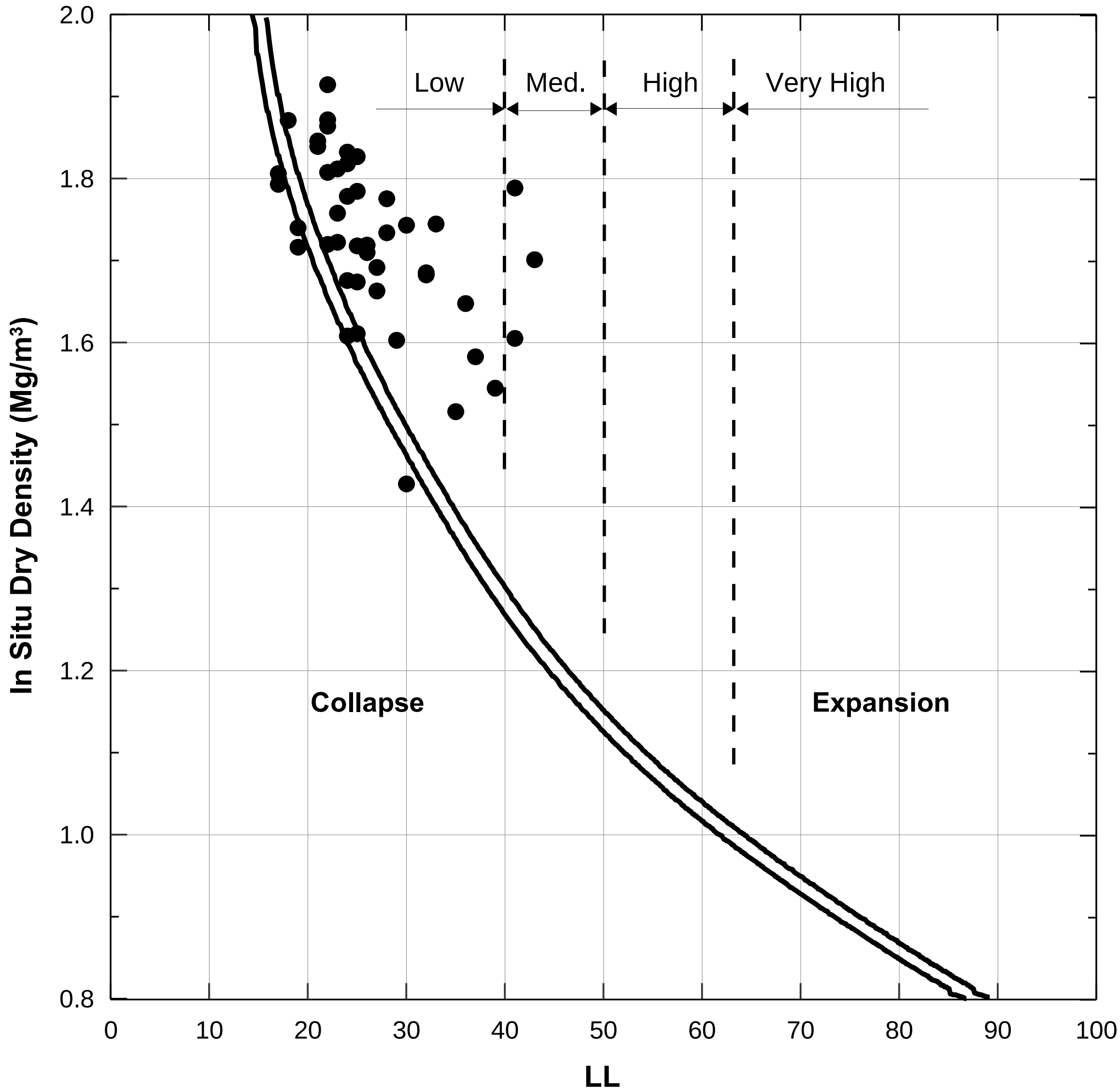

Figure 6.3-1
Guide to Collapsibility, Compressibility, and Expansion
(Mitchell and Gardner 1975, and Gibbs 1969)

E. Collapse

The soils encountered during the geotechnical investigation were not identified as collapsible based on the results shown on Figure 6.3-2. For bidding purposes, assume Existing Fill and Alluvial Fan will not be susceptible to collapse when saturated and no remediation of collapsible soil will be required.

F. Land Subsidence

The results of a preliminary assessment of land subsidence within the limits of CP1 indicates there has been no detectable land subsidence in the Fresno Area. For bidding purposes assume that subsidence from groundwater pumping is not an impact to the project area.

G. Corrosion

For buried concrete and steel elements, Caltrans Corrosion Guidelines (2003) consider a site to be corrosive if one or more of the following conditions exist for the representative soil and/or water samples taken at the site:

- Resistivity is 1,000 ohm-cm or less
- Chloride concentration is 500 parts per million or greater
- Sulfate concentration is 2,000 parts per million or greater
- pH is 5.5 or less

For bidding purposes, assume the Existing Fill and Alluvial Fan strata are considered non-corrosive to steel and concrete elements.

7.0 Design Considerations

7.1 Grade Separations

The CP1 alignment includes two below-grade track structures, the Fresno Grade Separation and the Jensen Trench. The Fresno Grade Separation is planned to be about 7,400 feet long with a maximum depth of about 55 feet to bottom-of-slab. The Jensen Trench is planned to be about 4,400 feet long with a maximum depth of about 17 feet to bottom-of-slab. Both below-grade track structures will require mass excavation in an urban environment.

This section discusses the considerations and design values that contributed to the choice of excavation support system, lateral loads, options for ground improvement, and lateral deflection requirements. Design criteria and other requirements for grade separations are covered in Section 03 30 01, Design and Installation of Grade Separations. A discussion of the assumptions and design criteria for the preliminary design is included in the U-Troughs, Bridges and Elevated Structures Report (URS/HMM/Arup 2011).

7.1.1 Support of Excavation

Widened and sloped excavations are not feasible at the Fresno Grade Separation because of right-of way and temporary easement restrictions. The preliminary design of the Fresno Grade Separation assumes internal bracing will be required. Sufficient right-of-way is available at the Jensen Grade Separation to preclude the need for shoring.

For preliminary design of the Fresno Grade Separation, a secant pile wall system was selected for temporary support in consideration of the sandy nature of the soils and the potential for a high groundwater table. At the time the excavation support system was selected, the groundwater conditions had not been verified by the 30% Geotechnical Investigation. A conservative groundwater table elevation of 10 feet below grade was assumed for the preliminary design. Sheet pile walls were not considered due to the indurated, cemented, or hardpan layers discussed in this report. Secant pile walls were also considered preferable over tangent pile walls and soldier-pile-and-lagging systems due to their stiffness and ability to limit lateral deflections and ground movements under the adjacent railroad and other nearby structures.

Results from the 30% Geotechnical Investigation suggest that soldier pile and lagging systems are appropriate for temporary excavation support based on the inferred strength of the ground and the current depth to groundwater. For design of temporary shoring, the baseline groundwater table can be assumed to be below the base of excavation.

For preliminary design, permanent shoring was selected in the vicinity of the Belmont Detention Basin, as shown on the Plans. The design groundwater table for permanent shoring walls along the CP1 alignment is 40 feet below grade.

The assumed sequence for construction of the Fresno Grade Separation is shown on the Plans. Design of the shoring system will need to consider staged bottom-up construction of the permanent walls while internal bracing is removed and the final permanent internal brace is installed at the top of the structure. Temporary support of the Dry Creek Canal and the north and south spurs of the SJVRR will also be required.

Selection and design of temporary excavation support systems is the responsibility of the Contractor. The design of such systems shall include all applicable surcharges including those imposed by construction equipment. Design and construction shall conform to the requirements of the Design Criteria Manual, Specification Section 31 15 13: Temporary Excavation Support and Protection, the Caltrans Trenching and Shoring Manual (2011), and the UPRR Guidelines Design and Construction of Shoring Adjacent to Active Railroads (2010).

7.1.2 Lateral Earth Pressures

Design lateral earth pressures for permanent structures are provided in the HST Design Criteria Manual. Lateral earth pressure coefficients for restrained and unrestrained permanent walls are provided in Table 7.1-1. Lateral earth pressure coefficients for the Existing Fill and Alluvium layers should be determined using the baseline effective friction angle (Φ’). Lateral earth pressure coefficients for on-site materials used as backfill should be determined using effective friction angle for the Alluvial Fan layer. Lateral earth pressure coefficients for off-site materials used as backfill can be determined using φ’ =34 degrees. Lateral earth pressure coefficients for temporary shoring may consider the effective cohesive strength of the Alluvial Fan layer, at the Contractor’s option.

Table 7.1-1
Earth Pressure Coefficients

Type of Wall	Lateral Earth Pressure Coefficients
Unrestrained Permanent Trench Wall	Active condition, $k_{\mathbf{a}} = \tan^{2}(45 - \Phi'/2)$
Restrained Permanent Trench Wall	At-rest condition $k_{\mathbf{a}} = 1 - \sin(\Phi')$

Peak ground accelerations are sufficiently low that seismic loads need not be considered. Surcharges from adjacent structures or facilities as well as impact/collision loads and UPRR railroad surcharge are to be considered per the HST Design Criteria Manual and Specification Section 03 01 30.

7.1.3 Ground Improvement

The subsurface soils encountered during the geotechnical investigation were predominately silts, sandy silts, and silty sands with SPT N_{60} blow counts above 25 and CPT cone tip resistances above 200 tons per square foot.

Due to the generally dense and stiff subsurface conditions, extensive deep ground improvement is not anticipated, except at river crossings or creek and channel locations where compressible soils are encountered. Ground improvement for track subgrades may be required where Existing Fill is encountered as described in Section 7.4.

7.1.4 Groundwater

The baseline design groundwater table depth for below-grade track structures is 40 feet BGL for the permanent structures. For construction, assume the baseline groundwater table is below the base of the excavation with the possibility of intermittent perched water conditions 10 feet BGL and deeper.

7.1.5 Lateral Deflection

Improvements adjacent to below-grade track structures, in some case, are likely to be sensitive to lateral deflections of shoring systems. Response values that trigger actions should deflections become excessive will vary per structure and stakeholder requirements.

The Contractor is responsible for evaluating each structure and utility which could be influenced by project construction and setting the response values. Allowable settlement or distortion of a building or utility varies depending on construction, age, materials, use, etc. These facilities must be evaluated on a case by case basis.

Temporary shoring and permanent U-trough walls should be designed to limit lateral deflections in accordance with the requirements of local jurisdictions, the Contract Documents, and AASHTO requirements.

7.2 Deep Foundations

7.2.1 Cast-in-Drilled-Hole Piles

The preliminary design includes deep foundations consisting of cast-in-drilled hole (CIDH) mono-piles and pile groups to support elevated structures and roadway overcrossings. The selection of CIDH piles was driven by large foundation loads and stringent deflection criteria. Right-of-way constraints and proximity of existing surface structures influenced the preliminary pile type and size selection to those with manageable pile cap footprints.

Driven piles were considered for roadway overcrossing bridge abutments.

7.2.2 Axial and Lateral Resistance

Axial resistances of CIDH piles are determined based on SPT N_{60} values. Baseline SPT N_{60} values provide the basis for estimating nominal skin friction, end bearing resistance, and p-y curves. Nominal resistances should be determined in accordance with Caltrans amendments to AASHTO requirements as per the HST Design Criteria Manual.

A significant consideration in the design of deep foundations must be given to lateral load resistance. This resistance is likely to be limited by the stringent deflection criteria necessary to maintain the track-structure interaction criteria. Typical spans and long-span elevated structures will exert large lateral demands on foundations potentially requiring additional piles for lateral resistance, enlarged pile caps, or post-tensioned CIDH piles.

7.2.3 Groundwater

Design of CIDH piles must consider the long-term possibility of groundwater fluctuations. The baseline design groundwater table depth for design of deep foundations is 40 feet.

7.2.4 Downdrag and Uplift Loads

Settlement adjacent to deep foundations can impose downdrag loads. However, soils along the alignment are generally of a consistency and type that is not conducive to time dependent behavior such as long-term consolidation settlements.

Downdrag loads can also be imposed by collapsible soils and settlements induced by seismic activity. However, the 30% Geotechnical Investigation did not identify soils or conditions that are susceptible to collapse or seismic deformations. For bidding purposes assume that any settlement of ground adjacent to deep foundations will occur during construction and that downdrag loads will be negligible.

Soils along the CP1 alignment are not considered sufficiently expansive to impose uplift loads that require consideration in the design of deep foundations. For the purposes of bidding assume uplift loads due to expansive soils do not need to be considered in the foundation design.

7.3 Retaining Walls

7.3.1 Wall Type Selection

The permanent retaining wall structures for the Fresno Grade Separation and Jensen Trench have been designed as cantilever and braced U-trough shaped structures. Permanent retaining walls for approaches to HST viaducts include conventional cast-in-place concrete walls and Mechanically Stabilized Earth (MSE)

walls. MSE walls are also anticipated at bridge abutments for roadway and pedestrian bridge overcrossings. MSE walls shall meet the requirements of Specification Section 31 38 13 Reinforced Slopes and Earth Structures.

Walls for roadway undercrossings (HST overcrossings) at Fresno Street, Tulare Street and other HST overcrossings shown on the Contract Documents are anticipated to be embedded walls and shall meet the requirements of Specification Section 31 66 16 on special foundation walls. Similar walls are anticipated for permanent shoring at the Fresno Grade Separation within 200 feet of the Belmont Detention Basin.

7.3.2 Structural Fill

Section 31 05 00 Common Work Results for Earthwork requires that Structural Fill have less than 15 percent fines. Figure 7.3-1 indicates approximately 15 percent of the soil sampled from the uppermost 50 feet in the vicinity of the Fresno Grade Separation will meet the specified fines content.

For bidding purposes assume 15 percent of the excavated materials exclusive of Existing Fill will meet Structural Fill requirements where adequate means and methods of separation are employed.

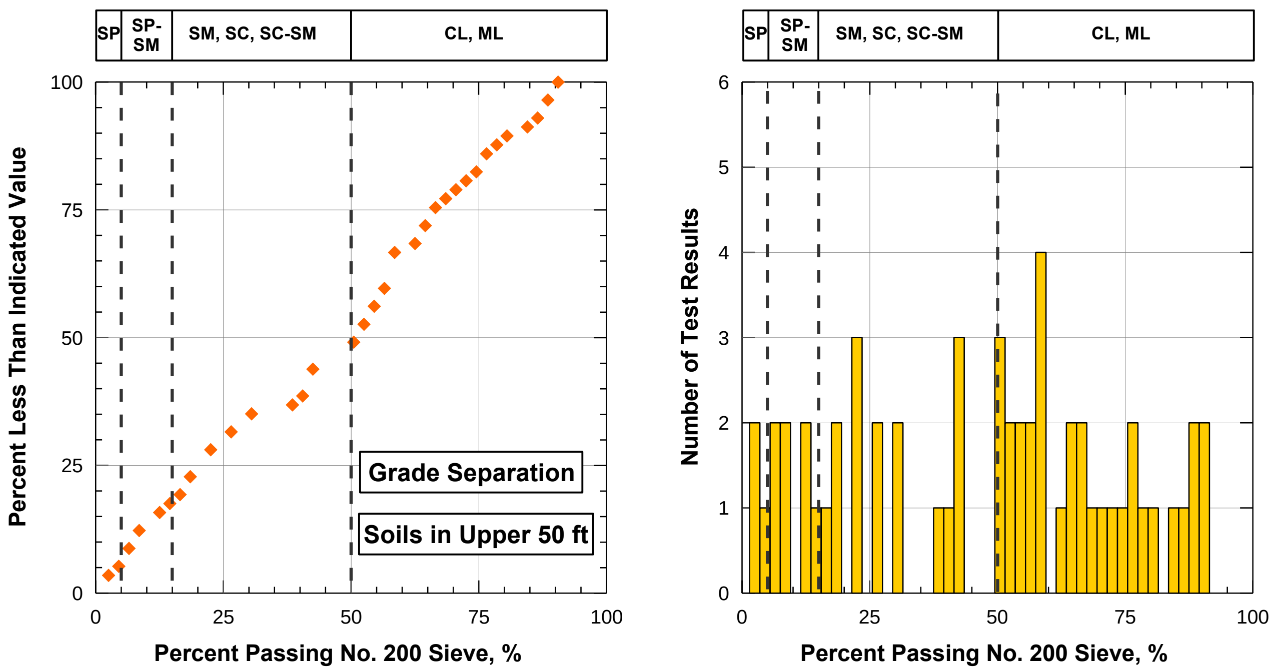

Figure 7.3-1
Fines Content Histogram

7.3.3 Lateral Deflections

Existing improvements and structures adjacent to special foundation walls, in some cases, are likely to be sensitive to lateral deflections. Threshold deflection values and response plans associated with excessive deflections will vary by structure and stakeholder requirements.

7.3.4 Drainage and Scour

The alignment between W Clinton Avenue and E Jensen Avenue crosses a floodplain. Adequate drainage is essential to the performance of retaining walls that are not designed for hydrostatic loads. Where walls

are backfilled with structural backfill, the backfill shall be considered free draining and additional drainage requirements (apart from the conventional weep holes and toe drains) are not required.

Special foundation walls and soil nail walls require geocomposite drainage strips to be placed in a manner that provides adequate drainage to relieve hydrostatic pressure. Special foundation walls required at the Belmont Detention Basin shall be designed for the full hydrostatic pressure of the baseline groundwater level in this area.

There are no significant bodies of water along the CP1 alignment that require special consideration for scour protection. For bidding purposes, assume that conventional embedment for frost protection will be adequate to protect against scour.

7.4 Embankments and At-Grade

7.4.1 Material Selection

Embankment materials consist of embankment fill, transition zone fills, structural fill, drainage layers, and geosynthetics. Embankment materials shall meet the suitability, gradation, and plasticity requirements of Specification Section 31 05 00 Common Work Results for Earthwork. Transition Zone materials are required where embankments support trackway approach structures. Transition Zone materials shall consist of structural fill mixed with cement to meet the strength requirements in Specification Section 31 05 00.

7.4.2 Subgrade Compressibility

Embankment foundation design should consider the potential for post construction settlement in the near-surface Existing Fill soils. Requirements for over-excavation or other remediation of soft or loose soils should be determined based on characterization of the subgrade and Existing Fill from future geotechnical investigations to be carried out the Contractor. Typical construction practice for embankment construction in areas of known Existing Fill is to excavate to firm or native conditions and backfill with material meeting fill and compaction requirements.

For bidding purposes, assume all Existing Fill is to be removed and replaced with suitable materials in accordance with the Contract Documents unless otherwise directed in the Design Criteria Manual.

7.4.3 Compaction Control

The Contractor shall provide quality control measures to ensure compliance with specified requirements. Embankment foundation and subgrade preparation and the placement and compaction of fills shall be performed under the surveillance of a California registered Geotechnical Engineer employed by the Contractor, as required by the Contract Documents.

7.4.4 Subgrade Preparation

Subgrade preparation shall meet the requirements of Specification Section 31 05 00. Subgrade preparation includes fine grading, reworking as necessary, and preparation of cut, fill, or embankment upon which the structure and equipment foundations, pipe, sub-ballast, sub-base, base, and pavement will be placed. Unsuitable subgrade material, such as weak or compressible soils, shall be removed. The entire surface of subgrade shall be scarified, moisture conditioned, and recompacted in accordance with the Contract Documents. Subgrade stabilization material shall be incorporated if required.

7.4.5 Drainage, Scour and Erosion

Where an embankment is located in a flood plain, the embankment design shall include slope protection consisting of a drainage layer and protection riprap. The drainage material shall be designed to comply

with Terzaghi's filter criteria as defined in the Specification Section 31 05 00. This layer should extend up to the highest flood water level plus additional freeboard as required by the Design Criteria Manual and be underlain by a layer of geosynthetic membrane.

In accordance with the Design Criteria Manual, the highest flood water level is the 100-year flood level. The predominance of the alignment between W Clinton Avenue and E Jensen Avenue is within the 100-year floodplain. For bidding purposes, a geosynthetic membrane, drainage layer, and rip-rap protection is required for all embankments over 5 feet high north of E Jensen Avenue.

7.5 Jacked Box Tunnel

The preliminary design has assumed the following key dimensions for the Jacked Box beneath SR 180:

- Total Length: 360 feet
- Jacked Length (excluding shield): 240 feet
- Thickness of the walls, roof, and base: 5 feet
- External width of the box: 53 feet
- External height of the box: 42 feet

The proposed right-of-way has been increased to 80 feet in the construction area of the Jacked Box. This allows a wider separation of the excavation shoring walls and allows sufficient working space for construction of the Jacked Box. As the excavation must be unbraced to allow space for constructing the Box, it is likely that the shoring walls will also be more substantial in this area than in other parts of the U-Trough. Due to the topography of the area and the slope of the alignment, the top of the Jacked Box projects above the ground level in the launch pit. It is expected that the Contractor will extend the shoring above the adjacent grade to permit the construction of overhead braces that clear the top of the box.

7.5.1 Ground Conditions

The Jacked Box Tunnel will be constructed in predominantly in Alluvial Fan deposits. The upper 4 to 5 feet of the tunnel excavation will likely encounter Existing Fill. Up to 11 to 12 feet of Existing Fill were placed below the existing SR 180 Caltrans bridge footings (see section 6.1.1). The nature of this fill is unknown but anticipated to have been placed in a controlled environment. Therefore, the Existing Fill over the Jacked Box Tunnel is anticipated to be more competent.

The ground must have sufficient strength to arch safely across the open cells and must accept the incremental advance of the shield into it without distress. Sometimes it is necessary to improve the ground in advance of tunneling. However, neither the Alluvial Fan nor the Existing Fill is anticipated to be susceptible to most conventional grouting techniques including permeation and chemical grouting.

For bidding purposes assume grouting ahead of the excavation is not possible.

7.5.2 Jacking Pit and Base Slab

The Jacked Box should be constructed on a base slab that is used to provide the reaction force against the jacks. This "jacking base slab" could be dowelled to the shoring wall to further distribute the jacking forces. Depending on the method used by the Contractor, the jacking base slab will be extended part way up the sides of the Jacked Box to provide lateral guidance and ensure it stays properly aligned in the early (critical) stages of the jacking operation.

The Jacked Box has been assumed to be a monolithic reinforced concrete section, though it is also possible that the Contractor may choose to divide the box into segments with "interstage" jacking between segments.

7.5.3 Anti-Drag System

An essential component of the box jacking system is the method by which drag from the structure is reduced. The anti-drag system (ADS) is designed to prevent this. The ADS prevents the overlying ground from being dragged along with box as it is jacked forward. In large embankments, there is some resistance to the drag force from the shear resistance of the embankment itself. However, this resistance may be insufficient to restrain the effect in the case of a wide box with low cover. If unrestrained, the ground on top of the box would be dragged forward, causing major disturbance and possible disruption to the overlying infrastructure.

The ADS wires do not isolate the sides of the box from the jacking force, so it is necessary to provide a method of reducing the frictional resistance of the sides to ensure that the force transmitted to the ground at the sides is minimized. Ground drag on the sides of the box is usually reduced by arranging the cutting edge so that a slightly larger hole is excavated than the box dimensions. Typically, the excavation is oversized by about 1 inch. However, the amount of over excavation has an effect on the amount of settlement that is seen at the surface, so overdig should be kept to the minimum necessary. Ground drag can also be reduced by lubricating the ground/structure interface with bentonite slurry. Usually both these methods are used together.

For bidding purposes, assume an ADS in conjunction with lubricating the ground/structure interface with bentonite slurry will be required.

7.5.4 Jacking Equipment

The jacking load will consist of the following:

- Reaction on shield structure
- Friction due to the dead load of the concrete structure on the ground/concrete launch portal
- Friction on the top and side of the concrete box against the soil

The total load, including all the components defined above, is estimated to be 18,500 kips. The maximum design single jack load is assumed to be 500 tons (1,000 kips), and the number of jacks is expected to be less than 20 units. For baseline purposes assume the jacking equipment should be designed for 18,500 kips.

7.5.5 Face Support

The excavation size proposed for this box is approximately 53 feet wide by 42 feet high. This would present a large excavation face that may be difficult to control. It is common practice to subdivide the excavation face into several compartments that can be excavated by mini-excavator or by hand. This method gives the ability to control the excavation by selectively excavating certain compartments at different rates in order to steer the box and maintain directional control. Some contractors also use doors that retain the face when not being excavated. These may be hydraulically controlled and linked to the main jacks to ensure a constant pressure is exerted on the face.

Adequate face support will be crucial to maintaining excavation stability in the event the Jacked Box encounters perched groundwater conditions.

7.5.6 Protection of Existing Structures

The jacked box tunneling operation must be carefully monitored and controlled to ensure the required performance and safety. Throughout the tunneling operation, movements at the ground surface over the area affected by the tunneling operation, jacking forces, and vertical and horizontal box alignment should be regularly monitored and compared to predicted or specified values.

Caltrans requirements for protection of their abutment and bridge are as follows:

- The abutment movements must not exceed ¼ inches horizontally ½ inches vertically
- The vertical deck movement must not exceed 1 inch for continuous superstructures and 2 inches for simple spans
- All proposals relating to crossing of the SR 180 will be subject to Caltrans review and approval before work is permitted to commence

The schedule assumed in bidding should consider sufficient time to complete the Caltrans design review process.

To meet these movement limitations, the Contractor shall assume that some form of compensation grouting of the ground under the abutment will be required and that compensation-grouting equipment will need to be linked to a settlement monitoring system to adjust the foundation of the bridge as jacking proceeds.

8.0 Construction Considerations

8.1 Regulatory Agencies

If temporary construction dewatering is utilized, a National Pollutant Discharge Elimination System (NPDES) permit from the Central Valley Regional Water Quality Control Board is required. In general, there is a long lead time required to obtain a NPDES permit. Refer to the Contract Documents for Storm Water Pollution Prevention Plan (SWPPP) requirements.

Gas detection and monitoring was not in the scope of the preliminary geotechnical investigation. It is the responsibility of the Contractor to investigate potentially gassy conditions that are likely present during construction of the Jacked Box Tunnel. Tunnel air monitoring and ventilation shall comply with OSHA 29 CFR 1926.800 regulations.

Trench excavations, shoring systems, sloped cuts, and other temporary structures shall comply with OSHA 29 CFR 1926.650 regulations.

8.2 Site Constraints

The Contractor shall conduct a site review to identify site specific-constraints which will impact the selection of construction sequence, equipment, and methods. Items affecting the selection of construction means and methods include, but are not limited to: (1) site accessibility and space restrictions; (2) restrictions on traffic disruption; (3) environmental concerns, including local restrictions on construction noise, vibration, and dust; (4) easement and right-of-way restrictions; and (5) location(s) of overhead and underground utilities and nearby structures.

8.3 Corrosive Soils

Laboratory soil corrosion and water chemistry testing conducted for 30% design did not indicate the presence of a corrosive subsurface environment.

8.4 Contaminated Soils

The ground investigation conducted for PEP indicated the presence of contaminated soils. The type, source, concentration, spatial extent, and variability of contaminants that may be encountered during excavation remains uninvestigated. However, because the project alignment follows existing freeway and railroad corridors portions of which are heavily industrialized, the Contractor shall expect to encounter contaminated soils during excavation pursuant to the baseline values presented in section 6.1.4.

A soil management plan and site-specific health and safety plan must be implemented prior to initiation of construction activities. If evidence of contaminated soil is found during excavation activities (e.g., stained soil, odors), soil sampling and testing will be required prior to any disposal or reuse. Refer to the Contract Documents for more information.

8.5 Difficult Excavation

CPTs performed for the PEP Geotechnical Investigation occasionally required pre-drilling at depths where cone penetrometers could not penetrate through hardpan layers. Specific CPT locations and depths where pre-drilling was required are indicated in the GDR. Pre-drilling extended between 5 and 15 feet in thickness, starting at depths ranging from 9 to 29 feet.

The Contractor shall expect that excavations will penetrate hardpan layers of variable thickness, hardness, and degree of cementation. Relatively thin, moderately hard, and moderately cemented hardpan layers may be excavated using heavy-duty equipment. However, thicker, harder, and/or more

strongly cemented layers may require specialized excavation equipment. Appropriate excavation equipment should be selected based on the anticipated variability in the hardpan layers likely to be encountered.

8.6 Groundwater Inflows

The baseline unconfined groundwater table is below the proposed excavation; however, there is a potential for perched groundwater to be present during excavation operations. The presence of perched groundwater during excavation may reduce the stability of excavated slopes and create unwanted softening or heaving of soils at the base of the excavation. In the event that shallow or perched groundwater conditions exist, appropriate dewatering techniques should be employed.

Likely dewatering systems consist of in-excavation sumps. Global dewatering schemes are not anticipated and shall be avoided due to impacts on adjacent structures.

8.7 Track Subgrade Improvement

Existing Fill was encountered in a number of boreholes along the CP1 alignment during the 30% Geotechnical Investigation. The Contractor shall anticipate variability in the thickness and suitability of Existing Fill for re-use. Deleterious material in the Existing Fill may include, but are not limited to, wood, glass, brick, metal, and coarse gravel. Existing Fill soils are likely suitable for re-use provided they satisfy quality requirements in terms of fines content, gradation, Atterberg limits, and electrochemical properties as required by the Contract Documents.

Soils along the alignment are relatively uniform and possibly suitable for the proposed HST track construction. However, unsuitable materials, such as soft clays, loose sands, and landfill debris are likely present at shallow depths at some isolated locations in this area. The geotechnical investigation conducted for this design stage is inadequate to characterize the presence and extent of these areas. Some soil improvement measures, such as lime treatment or over-excavation and replacement with materials is likely needed to improve the subgrade during the track construction.

8.8 Utilities and Other Obstructions

The Fresno to Bakersfield 15% Draft Utility Impact Report (URS/HMM/Arup 2012e) identifies nine High Risk Utilities, numerous Low Risk Utilities and 56 Special Utility Considerations. The Contractor is directed to this report for further information on the location and type of utilities at risk.

8.9 Deep Foundations

Deep foundations will be required to support the viaduct piers, retaining walls, and bridge abutments. There are a number of different issues which should be considered regarding deep pile foundations that are dependent on the type of pile being installed. The primary deep foundation types for this project include CIDH piles and driven piles.

8.9.1 Driven Piles

Due to the presence of very dense sand/silty sand layers at various depths throughout the project site, hard driving conditions may be encountered during installation of driven concrete piles. Piles may be subject to refusal if either the soil is too dense to accept the pile or the hammer energy is too low to drive the pile. The Wave Equation Analysis of Piles (WEAP) can be used to help select the proper pile driving equipment and predict drivability of piles. WEAP simulates and analyzes the dynamics of a pile under hammer impacts according to one-dimensional elastic wave propagation theories. The results are used to predict the dynamic compatibility of the hammer-pile-soil for evaluation of drivability of driven piles. Thus, it is useful to select equipment to safely install the pile to the desired depth and capacity.

As per Section 49-1.05 of the Caltrans Standard Specifications, undersize pre-drilling can be used to facilitate the pile driving in thick and dense sand layers. Pre-drilling holes shall not be greater than the least dimension of the piles. In addition, driven steel pile (open-ended pipe pile or H pile) can also be considered to penetrate layers with difficult driving conditions.

8.9.2 Cast-in-Drilled-Hole Piles

CIDH piles can be achieved in this region by a number of techniques which include drilling an open dry hole, drilling the hole with water, drilling the hole with a bentonite slurry, and drilling a temporarily cased hole. Each of these methods has their advantages and disadvantages. For baseline purposes assume CIDH piles will require temporary support to prevent caving given the granular nature of the soils.

Cobbles and boulders can impede drilling operations. Cobbles and boulders were not encountered during our exploration and are not anticipated as a baseline condition for construction of CIDH piles. Hardpan can also impede drilling operations and shall be considered in accordance with the baselines established herein.

8.10 Excavations

Shallow excavations will be required for the pile caps. Trenching may also be required for utility installation. For the shallow depth of these excavations, excavations may be cut vertically if the soils will "stand-up" without shoring.

In some areas the soils may be too loose or granular to achieve a 5-foot excavation and a sloped cut or bracing must be used in conjunction with falsework and engineered backfill. Backfill at sloped pile cap excavations must be compacted to provide sufficient lateral resistance.

Deep excavations will require shoring. Soils along the grade separation structures are generally strong but are prone to raveling and sloughing if left unsupported and exposed to the elements.

All slope design, bracing, and trench work shall meet the requirements of Federal and State OSHA Regulations. For bidding purposes assume the baseline behavior of Alluvial Fan will classify as OSHA Type B soils and the Existing Fill will classify as OSHA Type C soils.

Surface runoff on the site should be controlled so that it does not flow into open excavations. Surface runoff shall conform to standard SWPPP requirements.

8.11 Existing Structures

Existing structures include the following:

- McKinley Detention Basin
- Belmont Detention Basin
- 96-in diameter Belmont Storm Drain
- Union Pacific/BNSF Railroad
- North San Joaquin Valley Railroad Spur
- South San Joaquin Railroad Spur
- Dry Creek Canal
- SR 180
- SR 41
- SR 99
- Several irrigation canals south of E North Avenue.

8.12 Environmental Concerns

Noise and vibrations produced through the construction of the project structures should adhere to the project environmental management plan and comply with state and federal health and safety regulations.

Construction schedules shall consider earthwork to take advantage of dry season (April through October). Earthwork in the dry season must include provisions for dust mitigation in accordance with local and regional air quality regulations.

Requirements for erosion control are found in Specification Section 31 05 00.

8.13 Archeological Resources

Based on review of the F-B Archeological Survey (URS/HMM/Arup 2011) there are no historic properties or archeological resources within CP1.

8.14 Construction Consideration Matrix

Table 8.14-1 below has been prepared to capture, from an engineer's perspective, the site conditions that would be of concern to a bidding contractor. The list is not exhaustive, but identifies some conditions at each of the planned structures that could have cost implications when considered as part of the bid preparation.

Table 8.14-1
Construction Considerations Matrix

Location	Track Subgrade Improvement	Perched Groundwater Control	Groundwater Inflow	Difficult excavation - Hard Pan	Buried Utilities and Other Obstructions	Contaminated Soil	Compressible Soil	Collapsible Soil	Expansive Soil	Corrosive Soil	Regulatory Agencies	Deep Foundations	Temporary Shoring	Protection of Existing Structures	Environmental Concerns	Archeological Resources
At-Grade	X			X		X					X				X	
McKinley Avenue Roadway Overcrossing and Pedestrian Bridge		X		X	X	X					X	X		X	X	
Olive Avenue Roadway Overcrossing and Pedestrian Bridge		X		X	X	X					X	X		X	X	
Fresno Grade Separation	X	X	X	X	X	X					X		X	X	X	
Jack-Box Tunnel	X	X		X	X	X					X			X	X	
At-Grade	X			X		X					X				X	
Stanislaus Roadway Overcrossing		X	X	X	X	X					X	X		X	X	
Tuolumne Roadway Overcrossing		X	X	X	X	X					X	X		X	X	
Fresno Street HST Overcrossing	X		X	X		X					X	X		X	X	
Tulare Street HST Overcrossing												X				
Ventura Avenue Roadway Overcrossing and Pedestrian Bridge			X	X		X					X	X		X	X	
Church Avenue Roadway Overcrossing and Pedestrian Bridge			X	X		X					X	X		X	X	
Jensen Trench	X	X	X	X	X	X					X			X	X	
At-Grade	X			X		X					X				X	
Fresno (SR 99) Viaduct			X	X		X					X	X		X	X	
At-Grade	X			X		X					X				X	
Central Avenue Overcrossing	X	X		X	X	X					X	X		X	X	
E American Avenue Overcrossing	X	X		X	X	X					X	X		X	X	

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9.0 Instrumentation and Monitoring

Monitoring during underground construction is indispensable to verify assessments of ground behavior and make changes if necessary, and protect the Authority and the Contractor from any unsubstantiated claims by third parties who perceive that damage has occurred.

It is the Contractor's responsibility to survey and document the existing pre-construction conditions of adjacent structures and features – including railway structures, bridge structures, road embankments, and private roads and structures – and to monitor noise, vibration, and movements of these adjacent structures and features during construction in accordance with the Contract Documents and all applicable regional and local regulations.

The purposes of the instrumentation and monitoring program on this project are (a) to detect and monitor the vertical and horizontal movement of adjacent railway structures, bridge structures, road embankments, and private roads, and; (b) to comply with the requirements of major stakeholders (such as Caltrans in the case of the Jacked Box Tunnel below SR 180). The Contractor is responsible for developing a program that satisfies project objectives and meets contract requirements including monitoring, reporting, and implementing approved action plans should response (ground movement) values be exceeded.

Response Values consist of Threshold Values and Limiting Values. Threshold Values are set lower than Limiting Values and provide advance notification of ground movements that are trending towards damaging levels, such that mitigation measures can be employed prior to movements reaching critical levels. When a Limiting Value is approached or reached, an immediate suspension of excavation activities will be implemented until movements can be controlled and corrective measures put into place to prevent future damage. Response values will be determined based on the nature of work and are subject to the requirements of third parties including Caltrans and the adjacent railroads.

The instrumentation and monitoring plan shall ensure surface settlements are maintained within the allowable movements to prevent damage to existing structures, utilities and other facilities. At minimum, instrumentation shall include the following:

- Surface settlement points
- Inclinometers
- Tiltmeters
- Extensometers
- Crack Gauges
- Shoring monitoring points
- Utility monitoring points
- Building monitoring points

Of particular importance are the abutment foundations supporting SR 180 over the proposed Jacked Box Tunnel. The Contractor will need to satisfy all Caltrans requirements for monitoring of the abutment foundations and SR 180 during construction of the Jacked Box Tunnel. A "high-sensitivity" monitoring system possibly consisting of settlement sensor liquid tube fixed to the abutment, foundations and/or girders, capable of a continuous monitoring readout with real-time movement data shall be anticipated. Provisions for access to the abutment will need to be made throughout tunneling operations for instrumentation monitoring and maintenance purposes.

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11.0 Glossary

Artesian: A condition that exists when the water table piezometric surface lies above the ground level

Atterberg Limits: The water contents of a soil mass corresponding to the transition between a solid, semi-solid, plastic solid or liquid. Laboratory test used to distinguish the plasticity of clay and silt particles.

Boulder: A boulder is defined as a rock fragment that will not pass through a 12-inch (305mm) square opening, no matter how it is oriented in the opening. Boulder sizes are defined by the smallest size opening that the boulder can be oriented to pass through.

Bulk Unit Weight: The total weight of water and soil particles contained in a unit volume of soil.

Bulking/Swell factor: (volume of soil after excavation) / (volume of soil in-situ)

Cobbles: Soil particles between 3 inches (76 millimeters) and 12 inches (305 millimeters) in size.

Cohesion: The force that holds together molecules or like particles within a substance.

Cohesionless soils, Non-cohesive soils: Granular soils (silt, sand, and gravel type) with no shear strength unless confined.

Cohesive running: See Running.

Cohesive soils: Contains clay minerals and possesses plasticity.

Confined aquifer: An aquifer which groundwater is confined under pressure which is significantly greater than atmospheric pressure.

Consolidation: Reduction in soil volume due to squeezing out of water from the pores as the soil comes to equilibrium with the applied loads.

Deep wells: Wells with pumps installed at depth.

Depressurization: Reduction of water pressure by dewatering or other means.

Dewatering: The removal of groundwater to reduce the flow rate or diminish water pressure. Dewatering is usually done to improve conditions in surface excavations and to facilitate construction work.

Dry Unit Weight: The weight of solids (soil grains) to the total unit volume of soil. Units lb/ft^3, kN/m^3.

Structural Fill: Soils used as fill, such as retaining wall backfill, foundation support, dams, slopes, etc., that are to be placed in accordance to engineered specifications. These specifications may delineate soil grain-size, plasticity, moisture, compaction, angularity, and many other index properties depending on the application.

Firm, firm ground: Soil that remains stable in walls and face of an opening without initial support for sufficient time to permit installation of final support.

Flowing, flow, flowing ground: Soil that moves like a viscous liquid into an excavation.

Grain Size Distribution, Particle Size Distribution: Soil particle sizes that are determined from a representative sample of soil that is passed through a set of sieves of consecutively smaller openings.

Groundwater: Water that infiltrates into the earth and is stored in the soil and bedrock within the zone of saturation below the earth's surface.

Grout: A fluid mixture of water, cement, and/or sand, or of various additive chemicals that is injected directly into soil or voids. The fluid solidifies and hardens to fill voids and provide a water barrier and some reinforcement.

Hydraulic conductivity: See Permeability. The hydraulic conductivity is the volume flow rate of water through a unit cross-sectional area of a porous medium under the influence of a hydraulic gradient of unity, at a specified temperature. It is measured in units of cm/s, m/s or m/day and varies with temperature.

Hydraulic gradient: The difference in total head (piezometric levels), between two points in a hydraulic flow, divided by the length of the flow path (distance between the two points).

Hydrostatic head, hydrostatic pressure, pressure head: The height of a column of water required to develop a given pressure at a given point. Head may be measured in either height (feet or meters) or pressure (pounds per square inch, kilograms per square centimeter, or bars).

Jet grouting: Ground modification system utilizing injection under pressure of a cement-fluid grout mixture to mix grout with soil to develop an in situ material of enhanced strength.

K_0: The Coefficient of Earth Pressure at Rest (K_0) of a soil is the ratio of horizontal to vertical effective stress.

$$K_0 = \sigma_h'/\sigma_v'$$

Moist Unit Weight: Ratio between the total weight of soil including water, and the total volume of the soil.

Natural Water Content: The ratio between the mass of water and the mass of soil solids. w = (wet weight - dry weight) / dry weight.

Normalized Cone Resistance (Q_t): CPT tip resistance in a non-dimensional form and taking account of the in-situ vertical stresses.

$$Q_t = (q_t - \sigma_{v0})/\sigma_{v0}'$$

Normalized Friction Ratio (F_r): ratio, expressed as a percentage, of the sleeve friction (f_s) to the cone resistance (q_t) taking account of the in-situ vertical stresses.

$$F_r = [f_s/(q_t - \sigma_{vo})]100\%$$

Normalized CPT Soil Behavior Type (SBT_N): soil behavior type based on normalized cone resistance (Q_t) and normalized friction ratio (F_r).

Normally consolidated: A soil where the current effective overburden pressure is equal to the maximum overburden pressure.

Perched groundwater: An unconfined groundwater body in a generally limited area above the regional water table and is separated from it by a low permeability, unsaturated zone of bedrock or soil.

Permeability: The capacity of bedrock or soil to permit fluids to flow through it. See Hydraulic Conductivity.

Permeation grouting: Injection of cementitious or solution grouts into the pore space of granular soils.

Progressive failure: See Raveling.

q_c: CPT cone resistance

q_t: CPT cone resistance corrected for pore water effects, where A_n is the cone tip area ratio

$$q_t=q_c+u_2(1-A_n)$$

Raveling, Slow raveling, Fast raveling: Chunks or flakes of material drop out of the excavated surface due to loosening or to overstress and "brittle" fracture. In fast raveling ground, the process starts within a few minutes; otherwise, the ground is slow raveling.

Running, Cohesive Running ground: Granular soils that move freely into the excavated area. Granular materials without cohesion are unstable at a slope greater than their angle of repose (+ 30°-35°). When exposed at steeper slopes, they run like granulated sugar or dune sand until the slope flattens to the angle of repose. Cohesive running ground exhibits some apparent cohesion that exists from moisture content, weak cementation, and overconsolidation.

Shear Strength: The maximum shear stress which a soil can sustain under a given set of conditions. For clay, shear strength = cohesion. For sand, shear strength = the product of effective stress and the tangent of the angle of internal friction.

Shrinkage factor: (volume of soil after compaction) / (volume of excavated soil before compaction)

Spalling: See Raveling.

Specific Gravity: The ratio of the density of a body or a substance to the mass of an equal volume of water.

Spoil: The soil or rock materials generated during excavations. Included with these materials are drilling fluid, grout, waste cement or other construction related residue.

Squeezing ground: Soil that undergoes a time-dependent deformation near a tunnel as the result of load intensities which exceed the soils in situ strength. Ground squeezes or extrudes plastically into tunnel, without visible fracturing or loss of continuity, and without perceptible increase in water content. Ductile, plastic yield and flow due to overstress.

Standard Penetration Test, N-Value: Field test performed in general accordance with ASTM D 1586, Test Method for Penetration Test and Split – Barrel Sampling of soils. Test involves driving a 2-inch OD, 1.375 inch ID, split spoon sampler with a 140-lb hammer, falling freely from a height of 30 inches. The number of blows required to achieve each of three 6 inch increments of sampler penetration is recorded. The density of cohesionless or coarse-grained soils, and relative consistency of cohesive or fine-grained soils is defined as below:

Cohesionless Soils		Cohesive Soils	
N, SPT Blows/ft	**Relative Density**	**N, SPT Blows/ft**	**Relative Consistency**
0-4	Very loose	Under 2	Very soft
4-10	Loose	2-4	Soft
10-30	Medium dense	4-8	Medium Stiff
30-50	Dense	8-15	Stiff
Over 50	Very Dense	15-30	Very Stiff
		Over 30	Hard

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Swelling, Swelling ground: Soil that undergoes a volumetric expansion resulting from the addition of water. Swelling ground may appear to be stable when exposed, with the swelling developing later. Ground absorbs water, increases in volume, and expands slowly into the tunnel. Increase in soil volume; volumetric expansion of particular soils due to changes in water content.

u_2: pore pressure generated during cone penetration and measured by a pore pressure sensor just behind the cone.

Unconfined aquifer: An aquifer containing water that is not under pressure.

Unconsolidated: Loose sediment; lacking cohesion or cement.

Unified Soil Classification System (USCS): A system of soil classification based on grain size, liquid limit and plasticity of soils.

Appendix A
Soil Parameter Interpretations

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A1.0 Introduction

This appendix presents the results of statistical analyses used to develop the baseline soil parameters presented in Section 6.2 of the main report.

The purpose of this appendix is to enable bidders to evaluate the variability in ground conditions that may be anticipated during construction. Histograms and cumulative distributions have been prepared to present the range, mean, median, and standard deviation of data collected during this ground investigation. These interpretations are provided to illustrate the uncertainty associated with the estimates of baseline soil parameters.

The validity and reliability of the data presented herein have been reviewed, and in some cases, questionable data was excluded from the interpretations. Correlations used to derive soil parameters have been restricted to maximum reasonable values, based on engineering judgment.

Soil parameters have been measured and interpreted following TM 2.9.10 *Geotechnical Analysis and Design Guidelines*, in general accordance with Geotechnical Engineering Circular No. 5 (FHWA 2002) and AASHTO LRFD Bridge Design (2010) recommendations.

Cone penetration test (CPT) interpretations were based primarily on correlations published by Robertson (2009). All CPT data collected during the investigations was analyzed using the commercially available software CPeT-IT v1.7.3.35, developed by Geologismiki.

A2.0 CPT and SPT Correlations

A2.1 Total Unit Weight

2.1.1 CPT Correlation

Total unit weight was estimated from CPT results using the following correlation proposed by Robertson (2009):

$$\gamma_t = \gamma_w \cdot (0.27 \cdot \log(R_f) + 0.36 \cdot \log\left(\frac{q_t}{p_a}\right) + 1.236)$$

Where:

γ_w = Unit weight of water

R_f = Friction ratio

q_t = Cone resistance corrected for pore water effects

p_a = Atmospheric pressure

A2.2 Effective Friction Angle

2.2.1 CPT Correlation

Effective fiction angle was estimated from CPT results using the following correlation proposed by Robertson (2009):

$$\phi' = 29.5^2 \cdot B_q^{0.121} \cdot (0.256 + 0.336 \cdot B_q + \log Q_t)$$

(Applicable for $0.10 < B_q > 100$)

Where:

B_q = Pore pressure ratio

Q_t = Normalized cone resistance

2.2.2 SPT Correlation

Effective fiction angle was estimated from SPT results using the following correlation proposed by Hatanaka and Uchida (1996) for sands:

$$\phi' = \sqrt{15.4(N_1)_{60}} + 20°$$

Where:

$(N_1)_{60}$ = SPT N-value corrected for overburden and field procedures

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A2.3 Standard Penetration Test Blow Count

2.3.1 CPT Correlation

SPT N_{60} was estimated from CPT results using the following correlation proposed by Robertson (2009):

$$N_{60} = Q_t \cdot \frac{1}{10^{1.1268 - 0.2917 \cdot I_c}}$$

Where:

Q_t = Normalized cone penetration resistance

I_c = Soil Behavior Type Index

Given By:

$$I_c = [(3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2]^{0.5}$$

Where:

F_r = Normalized Friction Ratio

2.3.2 SPT Correction

The SPT correction for field procedures was applied as follows:

$$N_{60} = C_E N_{SPT}$$

Where:

N_{SPT} = Uncorrected field SPT N-value

C_E = Correction factor for Energy Ratio (ER) as measured in the field = ER/60

The SPT correction for overburden was applied as follows:

$$(N_1)_{60} = C_N N_{60}$$

Where:

N_{60} = SPT N-value corrected for hammer energy

C_N = Stress normalization parameter calculated as $C_N = \left({P_a}/{\sigma_{v0}'} \right)^n$

σ'_{vo} = In situ vertical effective stress

n = Stress exponent (assumed to be 0.5 for sands)

A2.4 Cone Tip Resistance

The cone tip resistance used for the statistical analyses refers to the static cone resistance q_c measured from cone penetration tests, as follows:

$$q_c = \frac{Q_c}{A_c}$$

Where:

Q_c = Force acting on the cone

A_c = Projected area of the cone

A2.5 Soil Modulus

2.5.1 CPT Correlation

Soil Modulus was estimated from CPT results using the following correlation (after AASHTO 2010):

$$E_s \cong 2q_c \text{ (Expressed in tons per square foot)}$$

2.5.2 SPT Correlation

The SPT correlation for Soil Modulus was applied using the elastic constants provided in Table A5.2-1 (after AASHTO 2010). For the purposes of interpretations, all soils were considered to be Category 2 soils.

Table A2.5-1
SPT Correlation to Soil Modulus by Soil Type

Category	Soil Type	Soil Modulus (tsf)
1	Silty, sandy silts, slightly cohesive mixtures	$4(N_1)_{60}$
2	Clean fine to medium sands and slightly silty sands	$7(N_1)_{60}$
3	Coarse sands and sands with little gravel	$10(N_1)_{60}$
4	Sandy gravels and gravels	$12(N_1)_{60}$

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A3.0 Grade Separation

The following sections present the results of statistical analysis performed on data obtained from boreholes and CPTs at the location of the proposed Fresno grade separation.

For the purposes of interpreting soil parameters at this location, the soil profile was analyzed in two layers: (1) upper 50 feet of soils (excluding Existing Fill) and (2) soils below 50 feet.

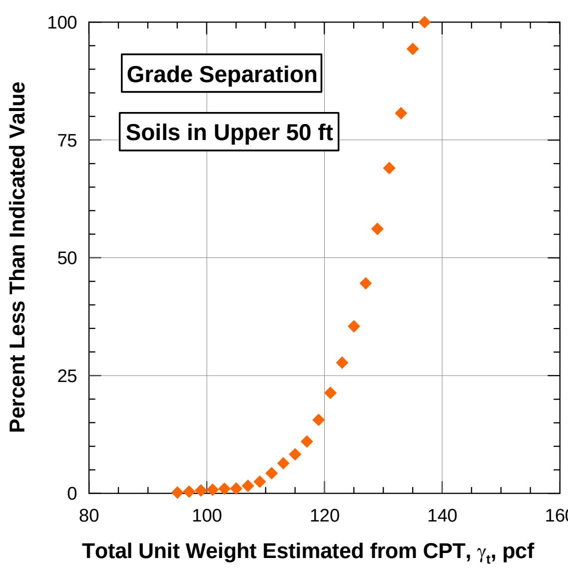
For each soil parameter, a supporting table has been provided to summarize the mean, median, standard deviation, and range of values obtained by soil layer and test type (e.g. CPT, SPT, or laboratory test).

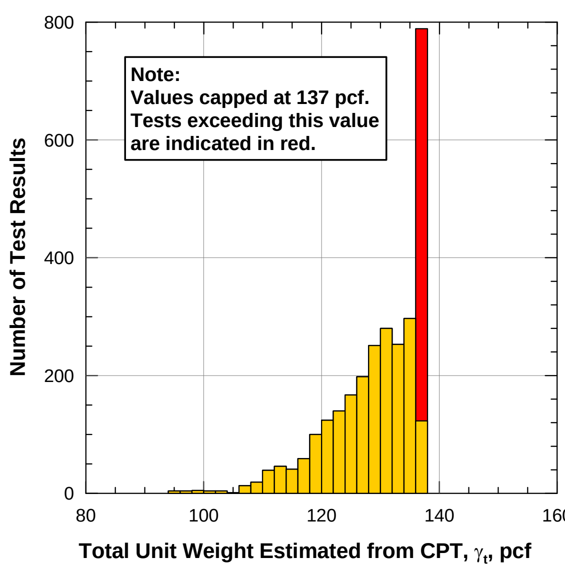
In some cases, soil parameters have been capped at a maximum value. Test results exceeding the maximum value are indicated in red on the histograms.

A3.1 Total Unit Weight

Table A3.1-1
Statistical Summary of Total Unit Weight – Grade Separation

Total Unit Weight	CPT	
	Upper 50 ft	Below 50 ft
No. Tests	2,172	1,525
Mean, pcf	127	130
Median, pcf	129	131
Standard Deviation, pcf	7	6
Maximum, pcf	137	137
Minimum, pcf	95	91





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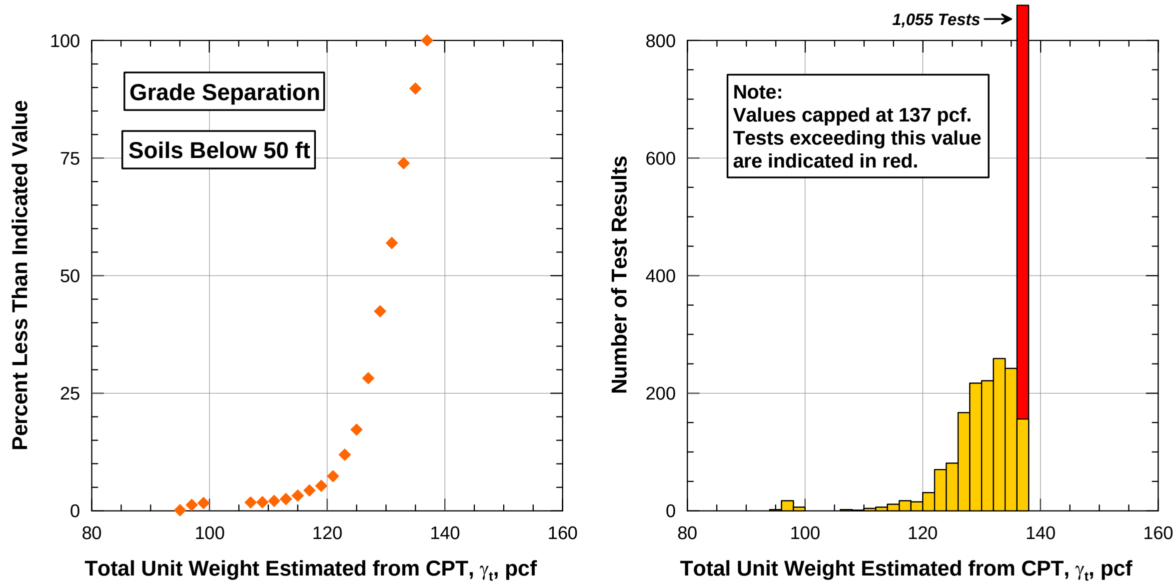

Figure A3.1-1
Statistical Summary of Total Unit Weight Estimated from CPT – Grade Separation

A3.2 SPT N_{60}

Table A3.2-1
Statistical Summary of SPT N_{60} – Grade Separation

SPT N_{60}	CPT		SPT	
	Upper 50 ft	Below 50 ft	Upper 50 ft	Below 50 ft
No. Tests	2,745	2,075	62	15
Mean, blows/ft	47	65	46	65
Median, blows/ft	45	64	42	65
Standard Deviation, blows/ft	20	18	24	15
Maximum, blows/ft	99	99	94	95
Minimum, blows/ft	5	17	5	44

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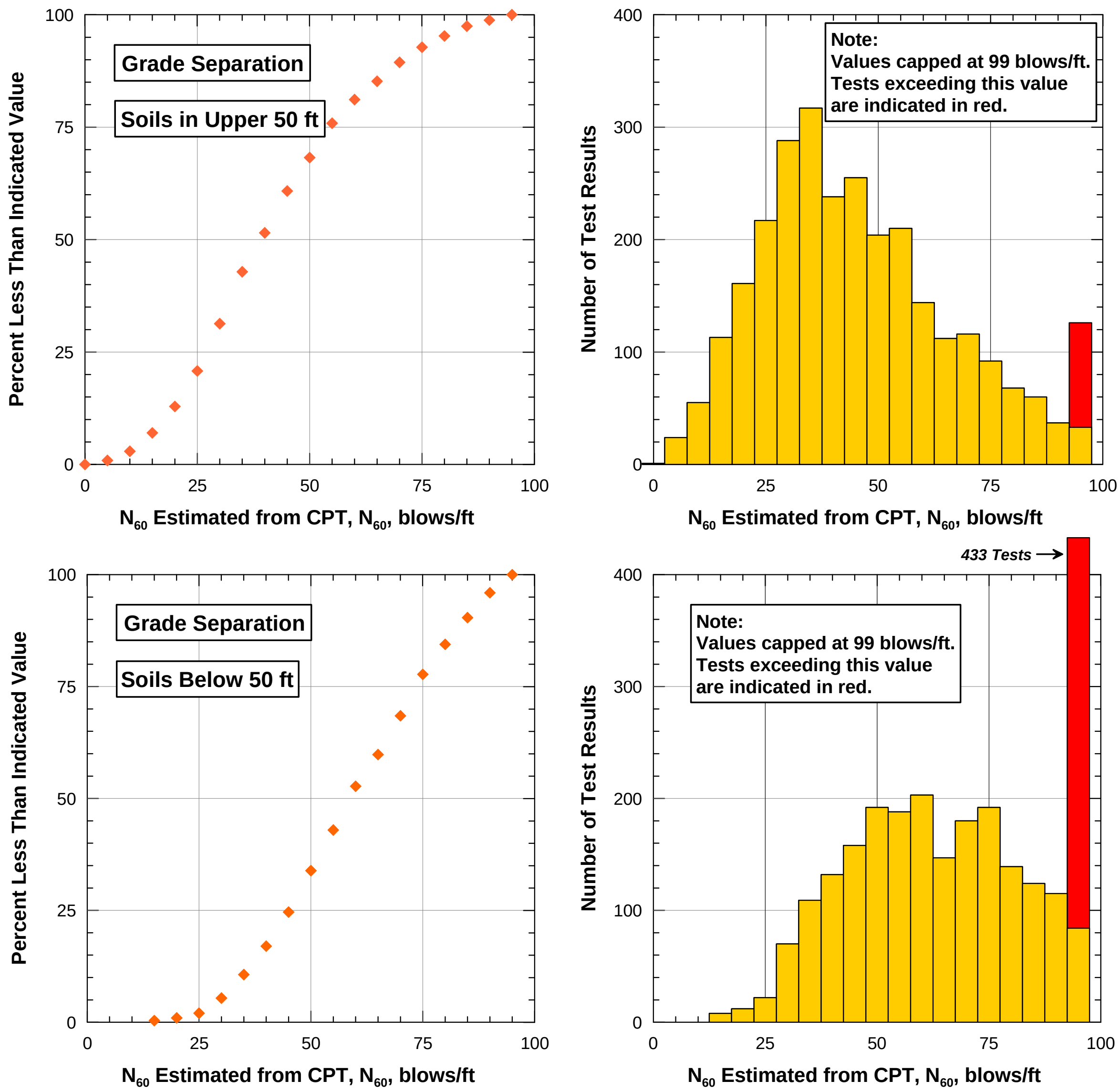

Figure A3.2-1
Statistical Summary of N_{60} Estimated from CPT – Grade Separation

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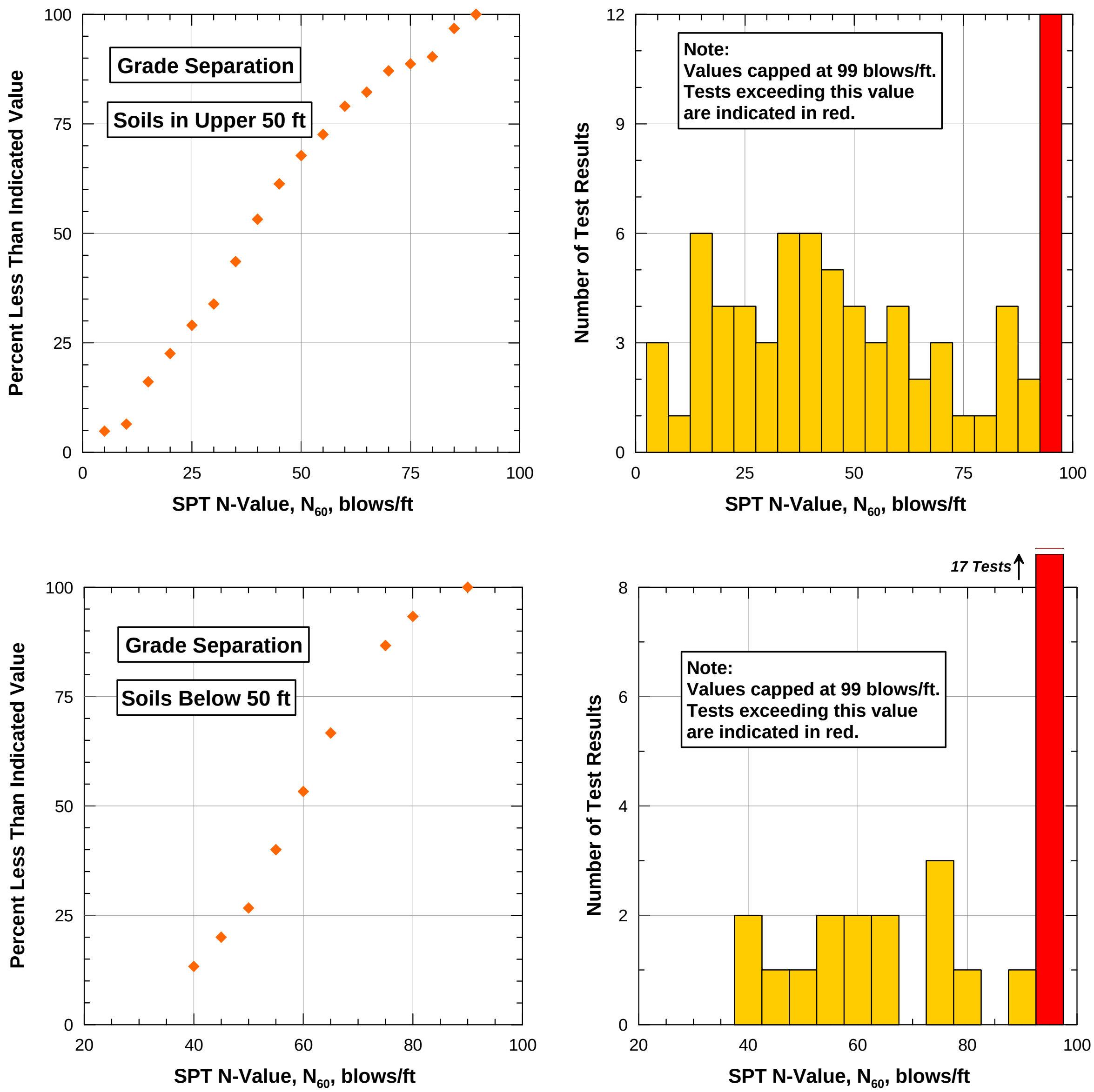

Figure A3.2-2
Statistical Summary of SPT N_{60}– Grade Separation

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A3.3 Effective Friction Angle

Table A3.3-1

Statistical Summary of Effective Friction Angle – Grade Separation

Effective Friction Angle	CPT		SPT		Laboratory	
	Upper 50 ft	Below 50 ft	Upper 50 ft	Below 50 ft	Upper 50 ft	Below 50 ft
No. Tests	2,683	1,935	176	15	11	1
Mean, deg	43	41	40	46	39	42
Median, deg	43	41	41	46	41	-
Standard Deviation, deg	2	2	6	4	3	-
Maximum, deg	51	49	50	50	43	-
Minimum, deg	34	34	20	40	32	-

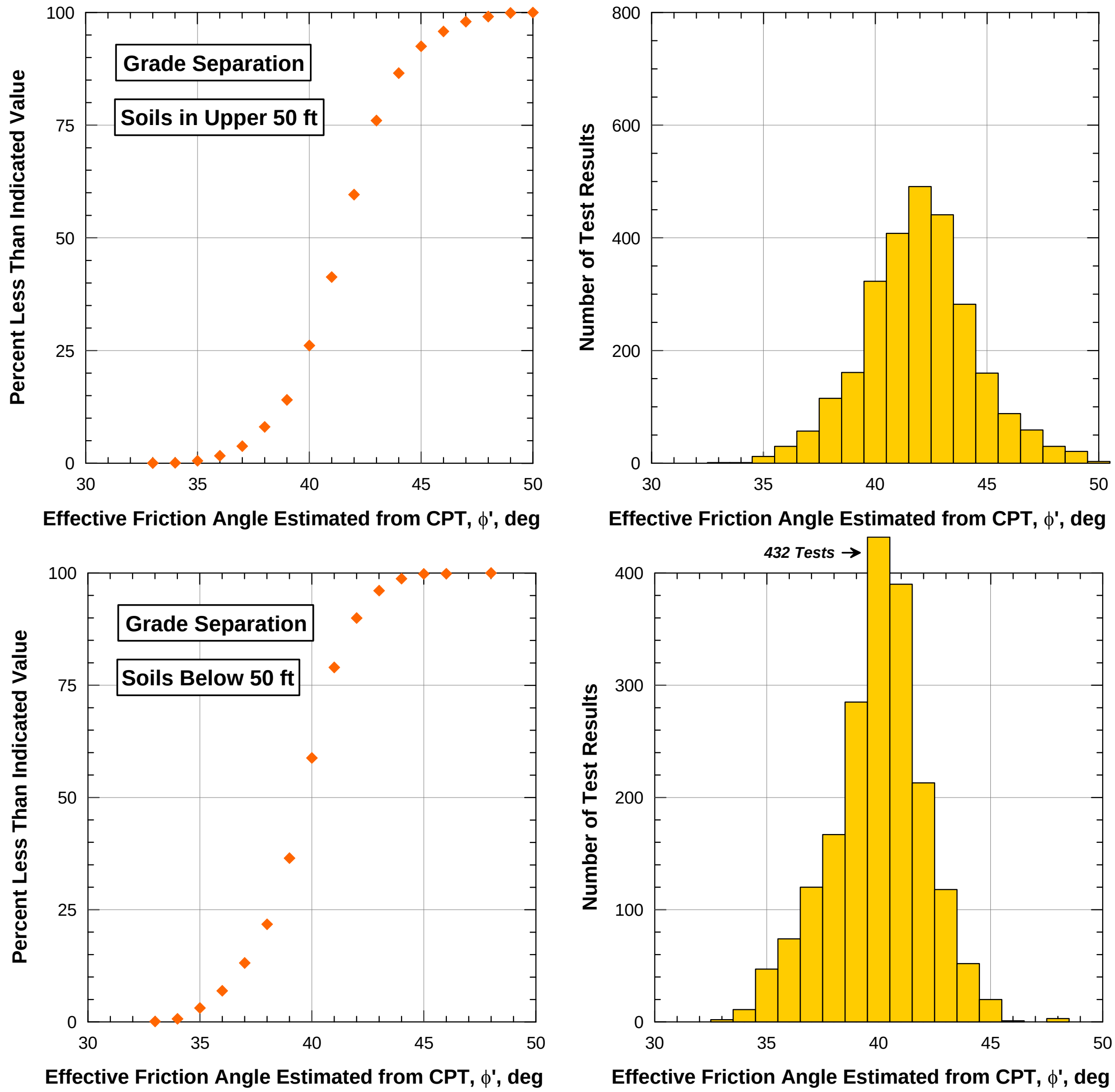

Figure A3.3-1

Statistical Summary of Effective Friction Angle Estimated from CPT– Grade Separation

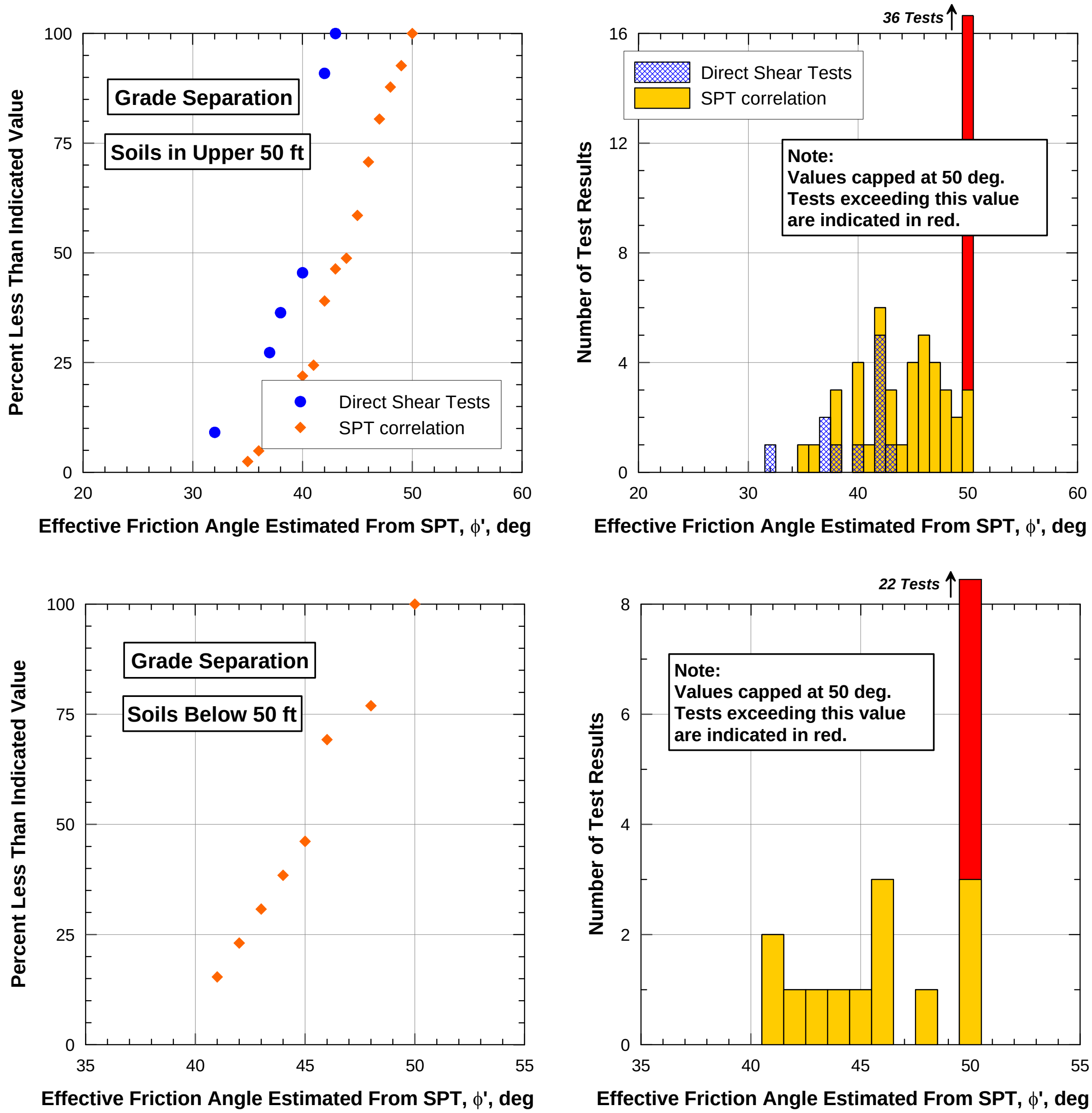

Figure A3.3-2

Statistical Summary of Effective Friction Angle Estimated from SPT– Grade Separation

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A3.4 Cone Tip Resistance

Table A3.4-1
Statistical Summary of Cone Tip Resistance – Grade Separation

CONE TIP RESISTANCE	CPT	
	Upper 50 ft	Below 50 ft
No. Tests	2,838	2,424
Mean, tsf	221	300
Median, tsf	194	285
Standard Deviation, tsf	123	138
Maximum, tsf	998	937
Minimum, tsf	17	34

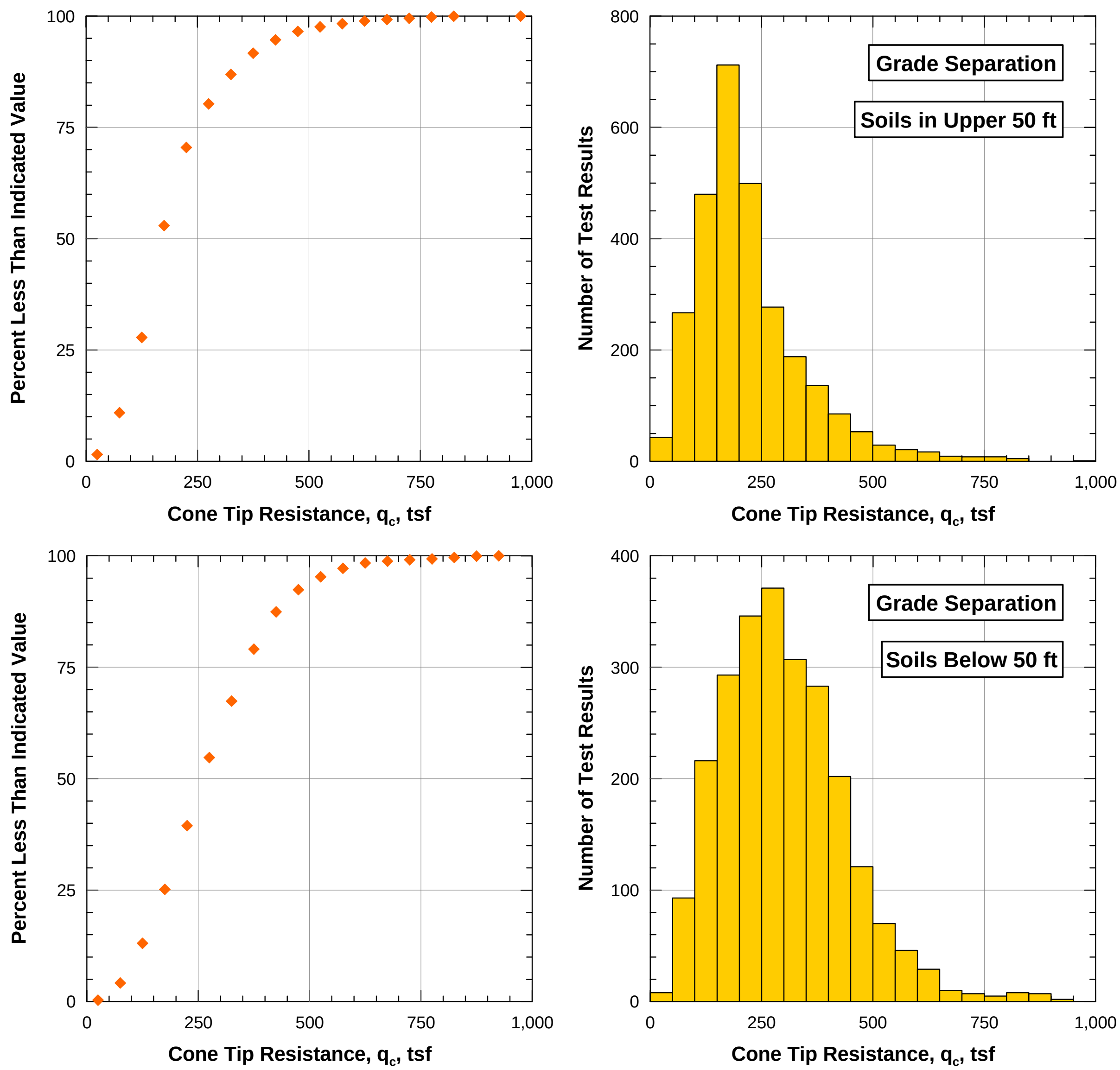

Figure A3.4-1
Statistical Summary of Cone Tip Resistance from CPT – Grade Separation

A3.5 Soil Modulus

Table A3.5-1
Statistical Summary of Soil Modulus Estimated from SPT– Grade Separation

Soil Modulus	SPT	
	Upper 50 ft	Below 50 ft
No. Tests	57	32
Mean, tsf	339	386
Median, tsf	305	417
Standard Deviation, tsf	155	97
Maximum, tsf	681	505
Minimum, tsf	96	186

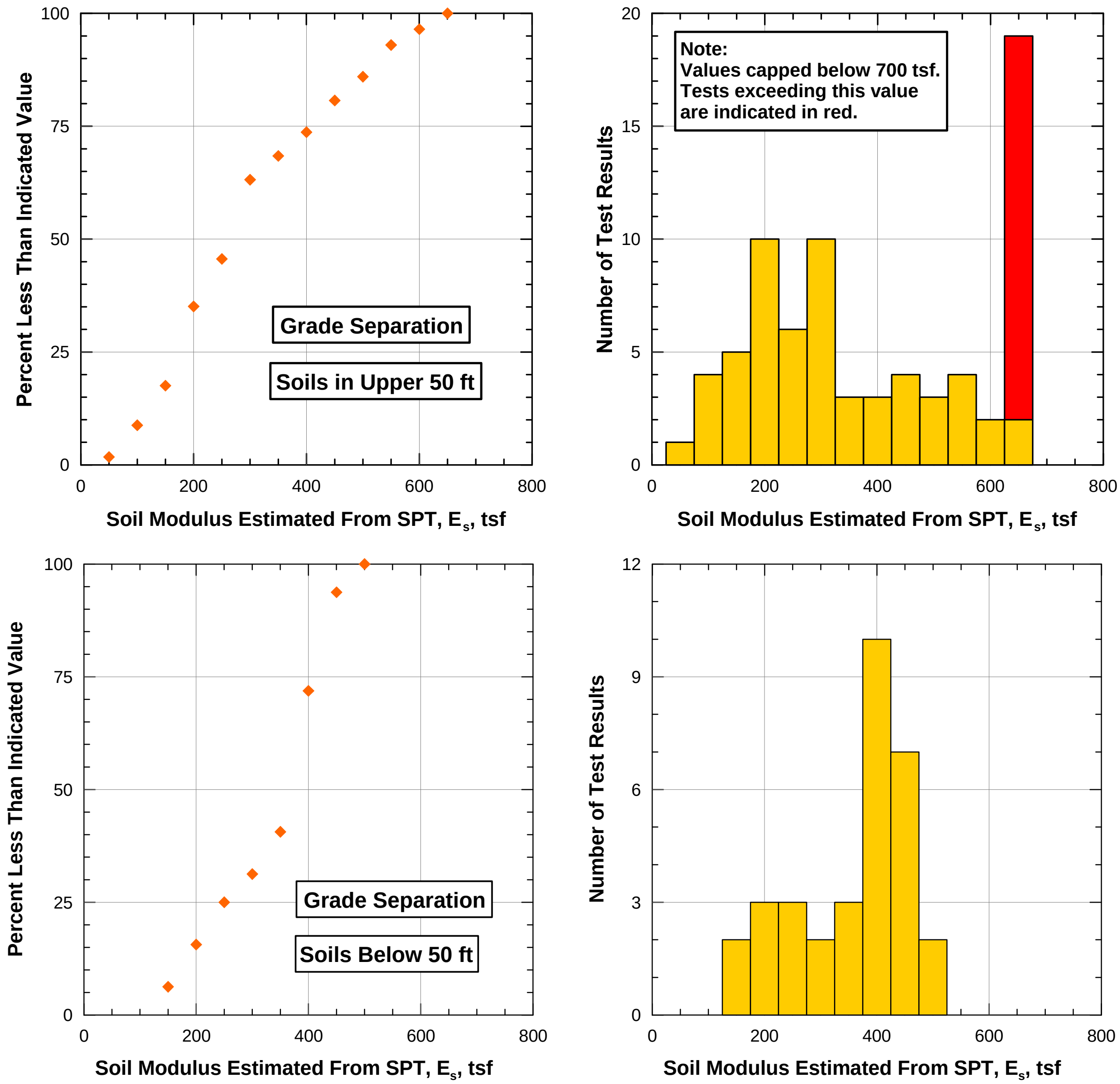

Figure A3.5-1
Statistical Summary of Soil Modulus Estimated from SPT – Grade Separation

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Table A3.5-2
Statistical Summary of Soil Modulus Estimated from CPT– Grade Separation

Soil Modulus	CPT	
	Upper 50 ft	Below 50 ft
No. Tests	2,745	2,075
Mean, tsf	419	532
Median, tsf	380	522
Standard Deviation, tsf	210	217
Maximum, tsf	1,692	1,686
Minimum, tsf	33	68

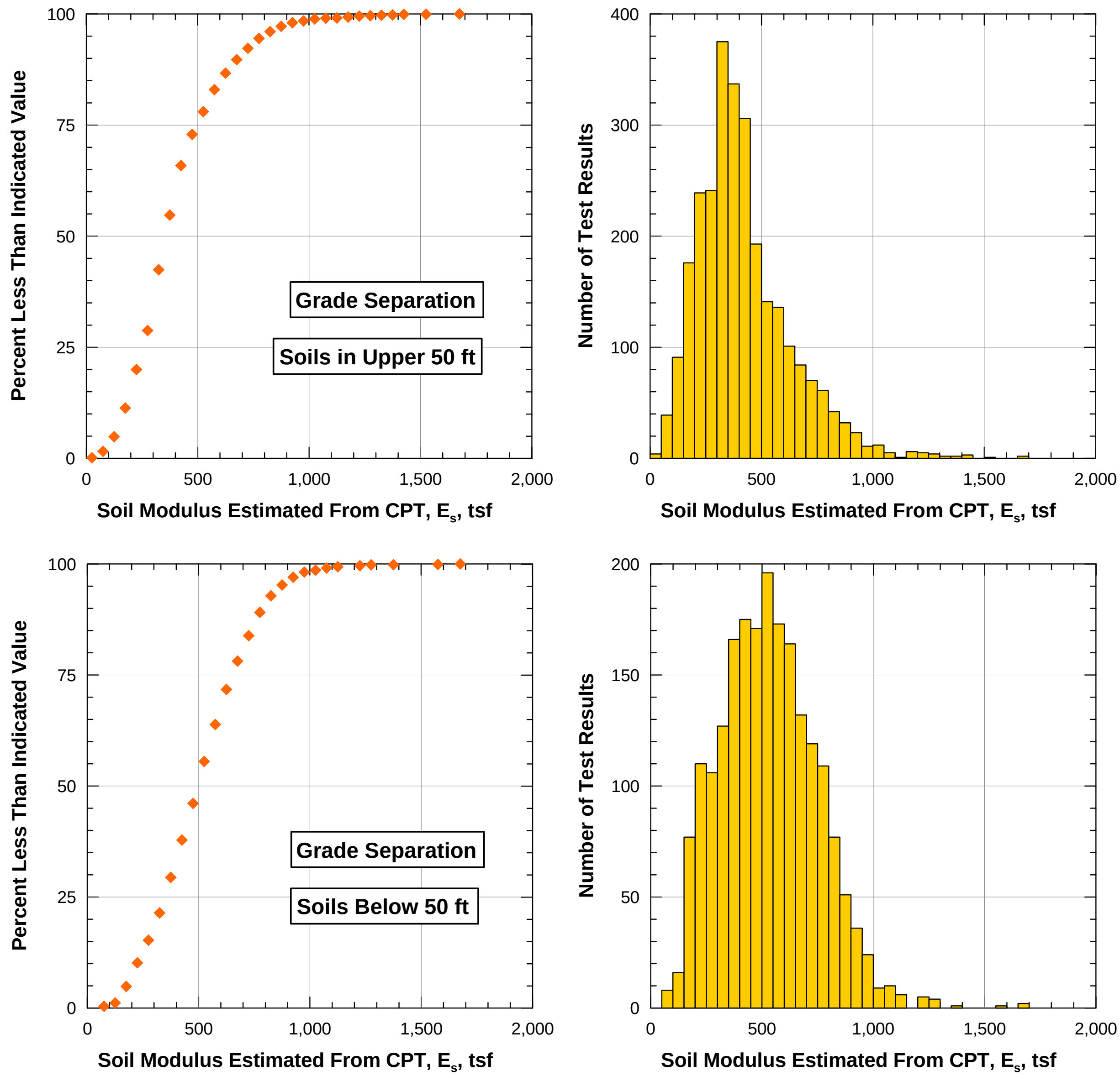

Figure A3.5-2
Statistical Summary of Soil Modulus Estimated from CPT – Grade Separation

A4.0 Jensen Trench

The following sections present the results of statistical analyses performed on data obtained from boreholes and CPTs at the location of the proposed Jensen Street Trench.

For the purposes of interpreting soil parameters at this location, the soil profile was analyzed in two layers: (1) upper 20 feet of soils (excluding Existing Fill) and (2) soils below 20 feet.

For each soil parameter, a supporting table has been provided to summarize the mean, median, standard deviation, and range of values obtained by soil layer and test type (e.g. CPT, SPT, or laboratory test).

In some cases, soil parameters have been capped at a maximum value. Test results exceeding the maximum value are indicated in red on the histograms.

A4.1 Total Unit Weight

Table A4.1-1
Statistical Summary of Total Unit Weight – Jensen Trench

Total Unit Weight	CPT	
	Upper 20 ft	Below 20 ft
No. Tests	288	646
Mean, pcf	122	130
Median, pcf	122	133
Standard Deviation, pcf	11	9
Maximum, pcf	137	137
Minimum, pcf	95	91

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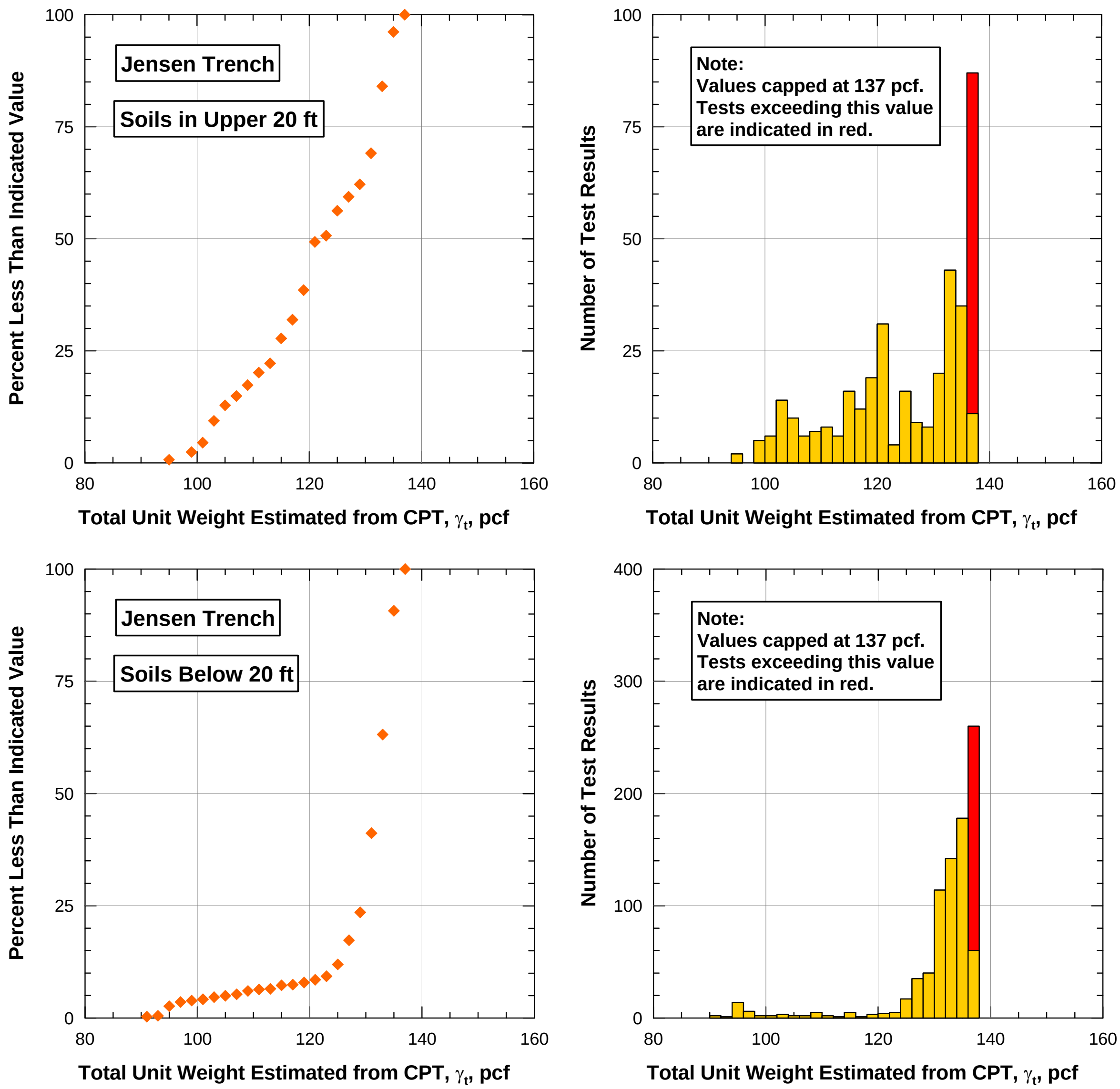

Figure A4.1-1
Statistical Summary of Total Unit Weight Estimated from CPT – Jensen Trench

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A4.2 SPT N_{60}

Table A4.2-1
Statistical Summary of SPT N_{60} – Jensen Trench

SPT N_{60}	CPT		SPT	
	Upper 20 ft	Below 20 ft	Upper 20 ft	Below 20 ft
No. Tests	359	813	24	27
Mean, blows/ft	38	52	27	59
Median, blows/ft	36	51	24	58
Standard Deviation, blows/ft	20	17	13	24
Maximum, blows/ft	99	99	50	99
Minimum, blows/ft	4	13	7	16

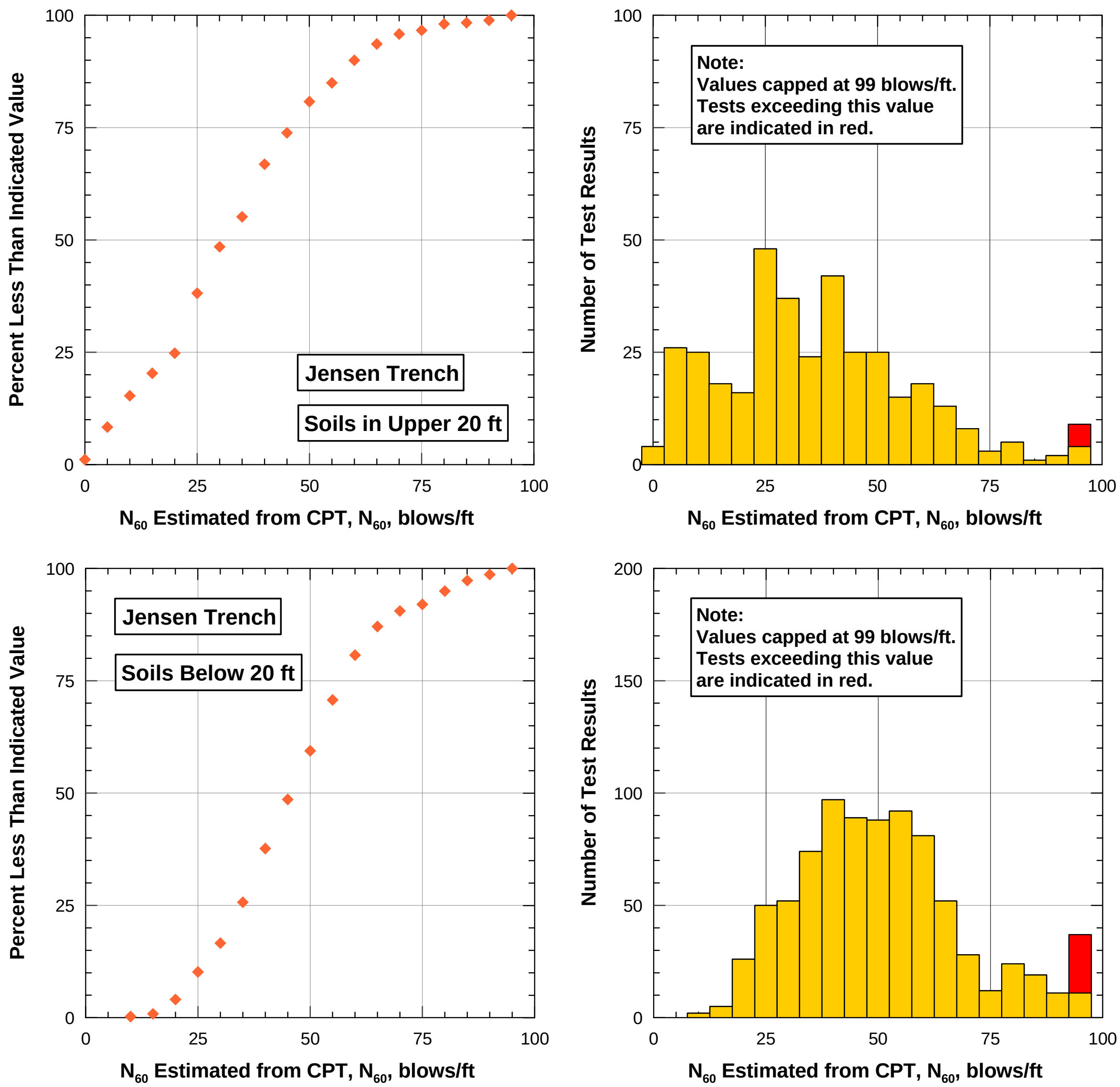

Figure A4.2-1
Statistical Summary of SPT N_{60} Estimated from CPT – Jensen Trench

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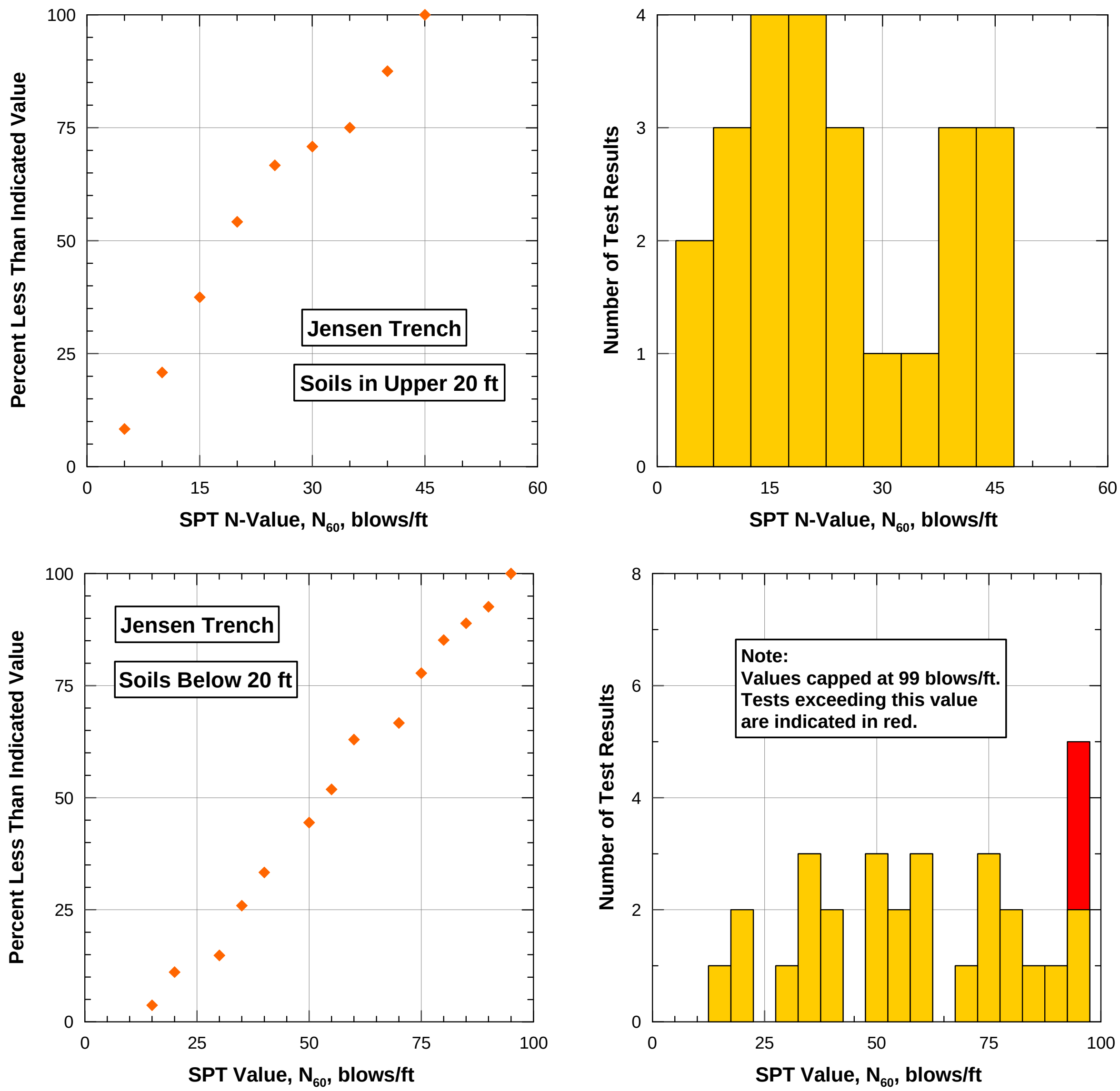

Figure A4.2-2
Statistical Summary of SPT N_{60} – Jensen Trench

A4.3 Effective Friction Angle

Table A4.3-1
Statistical Summary of Effective Friction Angle – Jensen Trench

Effective Friction Angle	CPT		SPT		Laboratory	
	Upper 20 ft	Below 20 ft	Upper 20 ft	Below 20 ft	Upper 20 ft	Below 20 ft
No. Tests	362	807	9	5	4	1
Mean, deg	44	42	45	44	37	41
Median, deg	44	42	46	44	37	-
Standard Deviation, deg	3	2	4	5	5	-
Maximum, deg	50	48	48	50	43	-
Minimum, deg	33	35	40	39	31	-

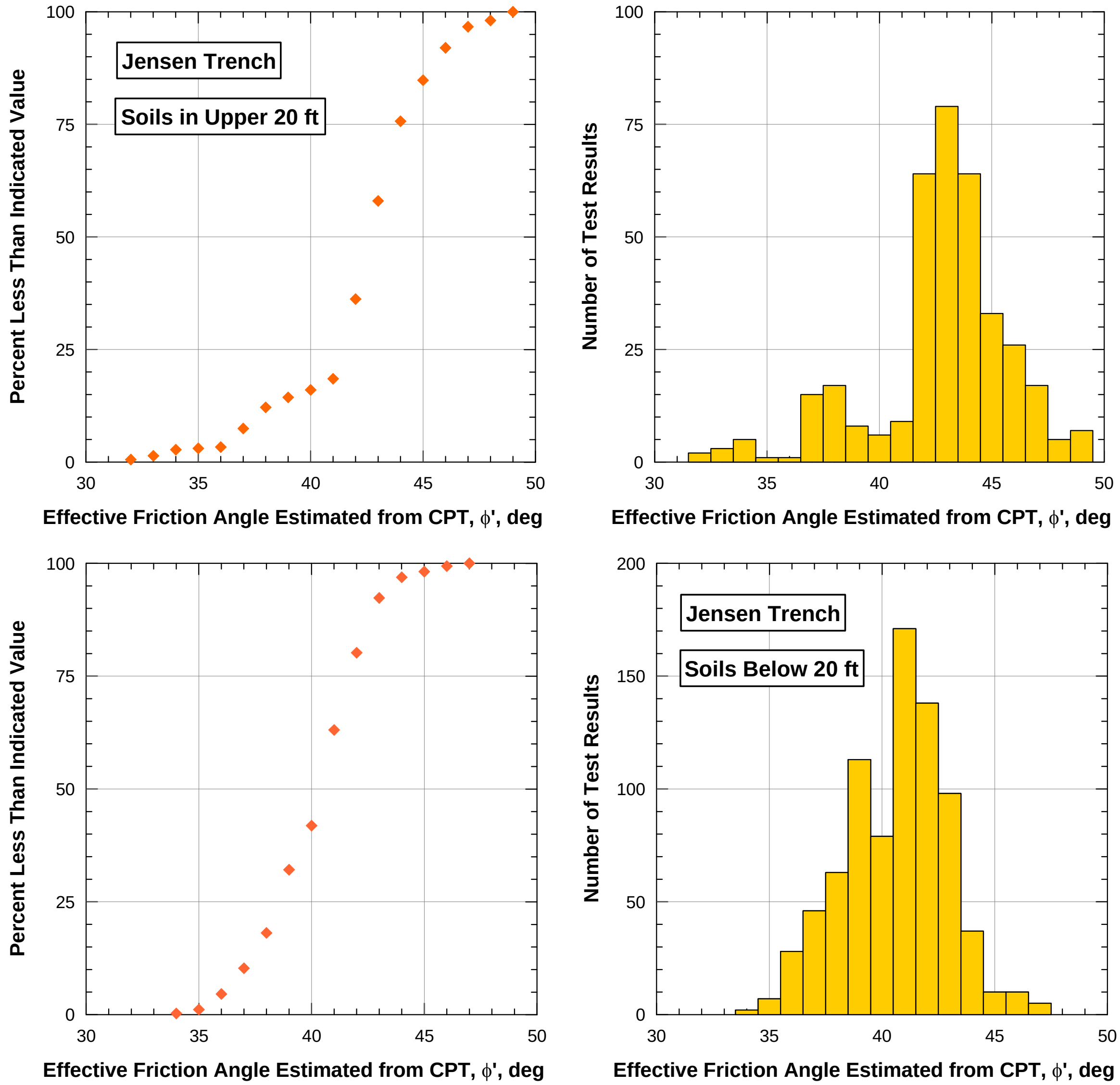

Figure A4.3-1
Statistical Summary of Effective Friction Angle Estimated from CPT – Jensen Trench

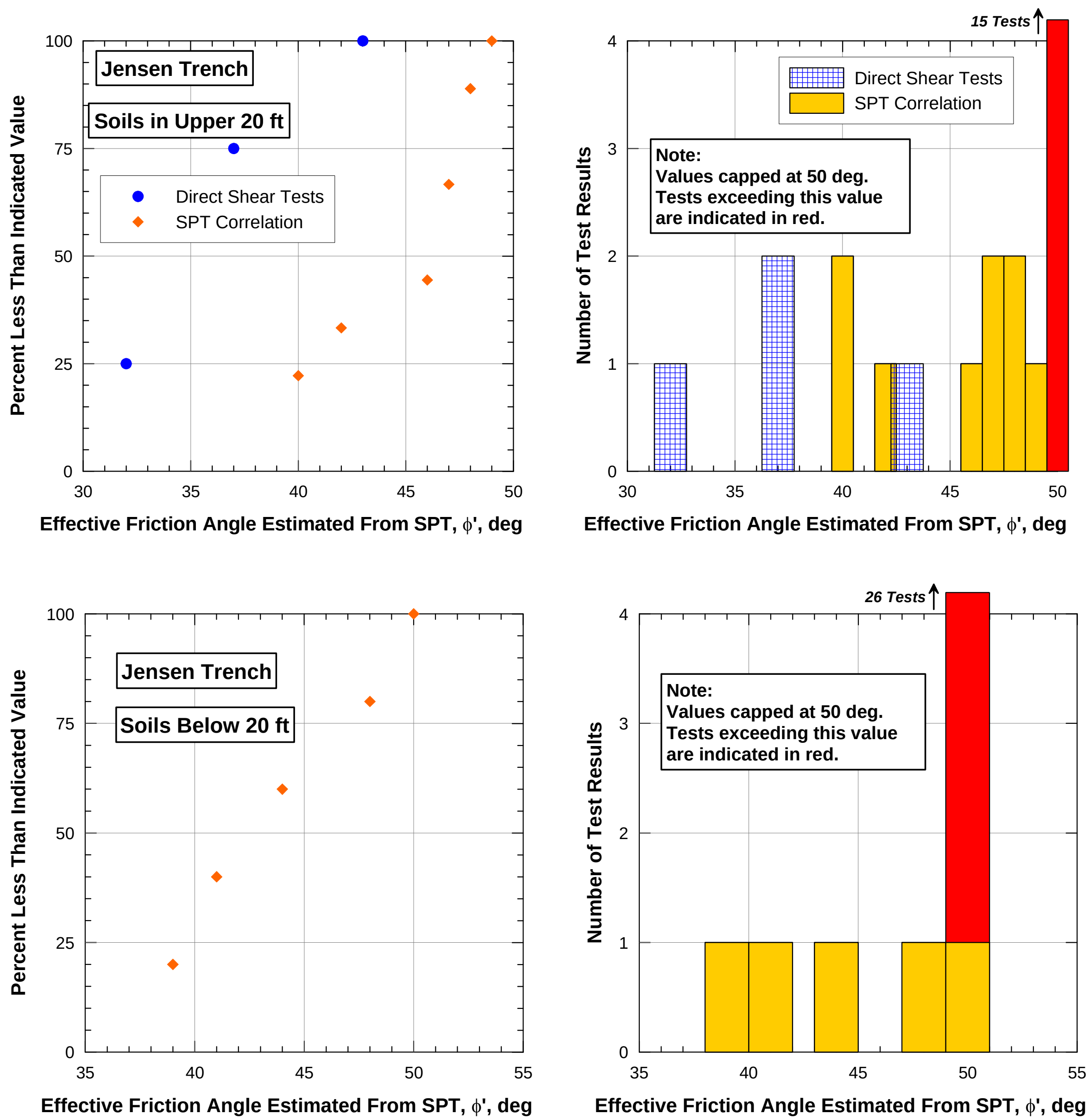

Figure A4.3-2
Statistical Summary of Effective Friction Angle Estimated from SPT – Jensen Trench

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A4.4 Cone Tip Resistance

Table A4.4-1
Statistical Summary of Cone Tip Resistance – Jensen Trench

Cone Tip Resistance	CPT	
	Upper 20 ft	Below 20 ft
No. Tests	364	846
Mean, tsf	178	209
Median, tsf	171	202
Standard Deviation, tsf	93	97
Maximum, tsf	538	657
Minimum, tsf	14	45

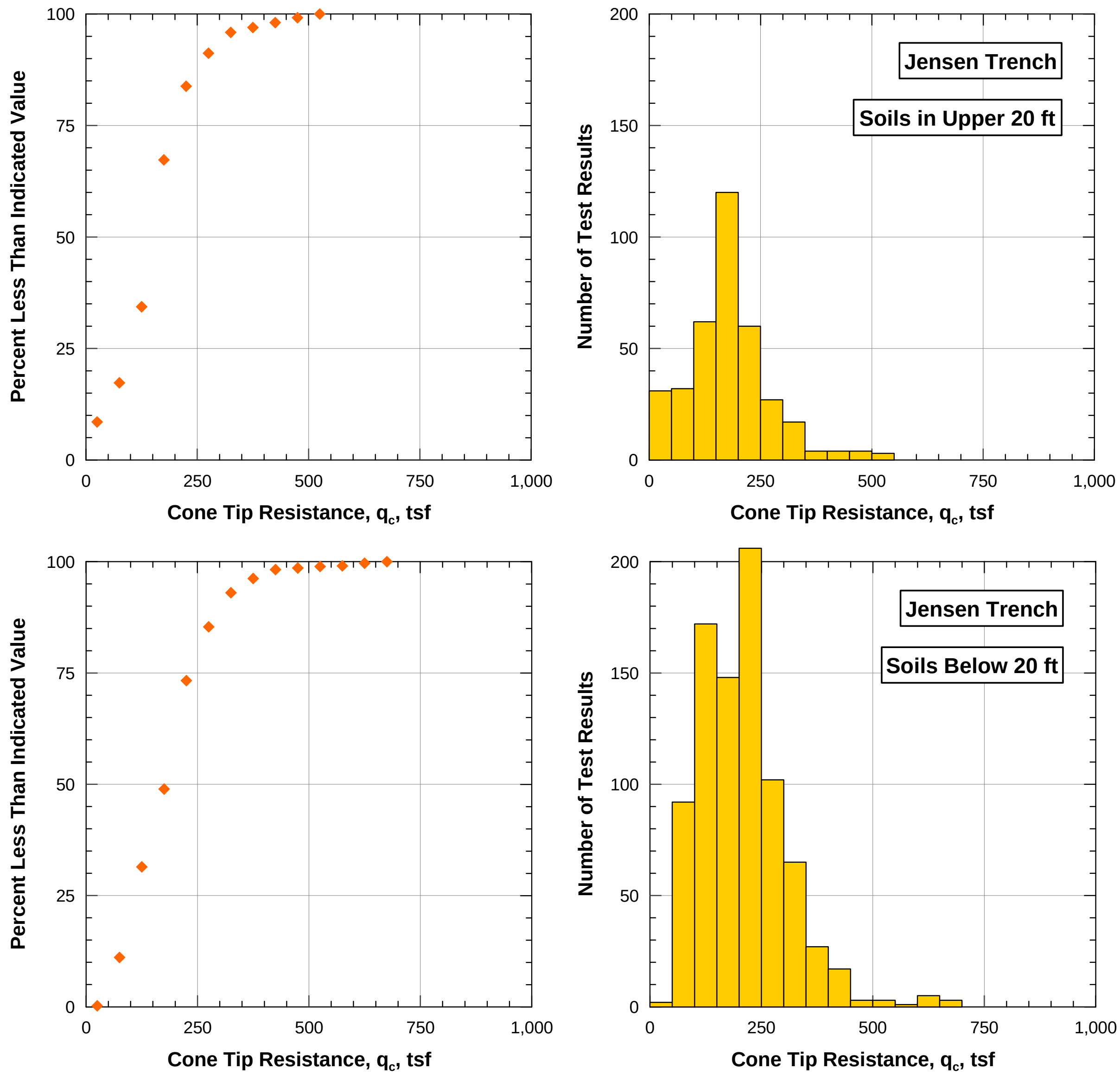

Figure A4.4-1
Statistical Summary of Cone Tip Resistance from CPT – Jensen Trench

A4.5 Soil Modulus

Table A4.5-1
Statistical Summary of Soil Modulus Estimated from SPT– Jensen Trench

Soil Modulus	SPT	
	Upper 20 ft	Below 20 ft
No. Tests	18	24
Mean, tsf	382	493
Median, tsf	409	511
Standard Deviation, tsf	120	145
Maximum, tsf	611	677
Minimum, tsf	177	161

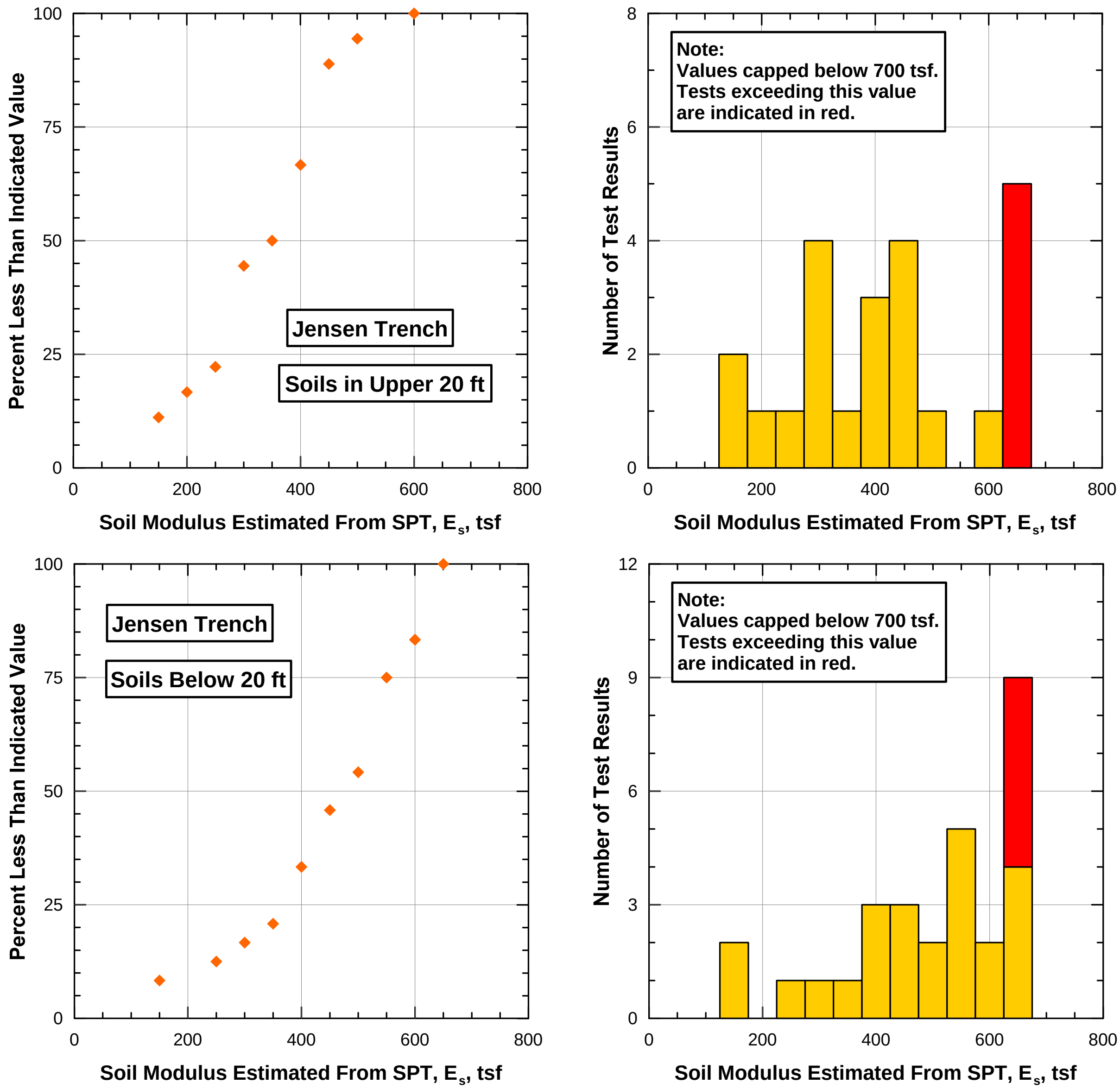

Figure A4.5-1
Statistical Summary of Soil Modulus Estimated from SPT – Jensen Trench

Table A4.5-2
Statistical Summary of Soil Modulus Estimated from CPT– Jensen Trench

Soil Modulus	CPT	
	Upper 20 ft	Below 20 ft
No. Tests	364	846
Mean, tsf	356	417
Median, tsf	343	404
Standard Deviation, tsf	186	193
Maximum, tsf	1,076	1,315
Minimum, tsf	29	90

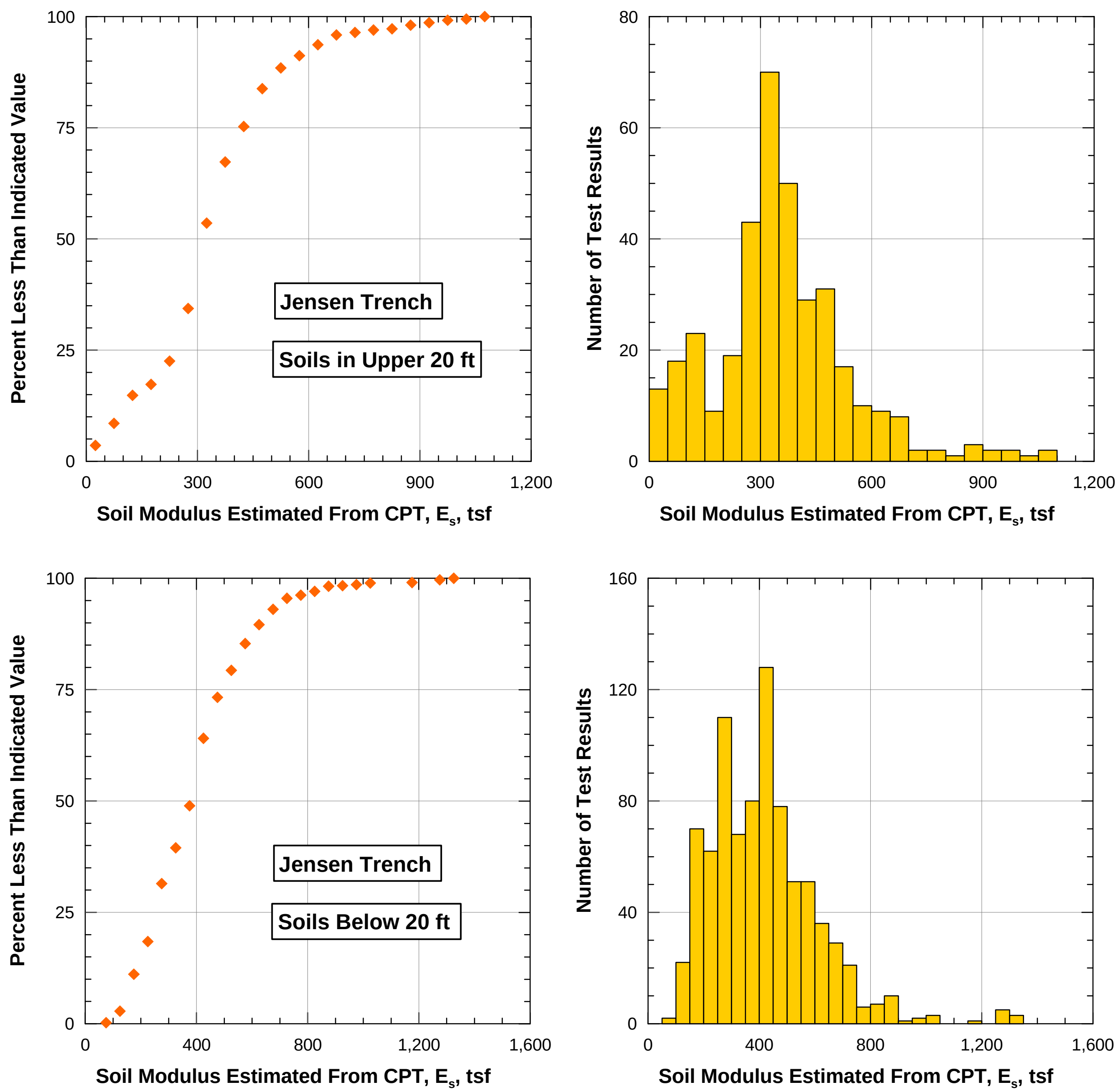

Figure A4.5-2
Statistical Summary of Soil Modulus Estimated from CPT – Jensen Trench

A5.0 Fresno Viaduct

The following sections present the results of statistical analyses performed on data obtained from boreholes and CPTs at the location of the proposed SR 99 (Fresno) viaduct.

For the purposes of interpreting soil parameters at this location, the soil profile was analyzed in two layers: (1) upper 60 feet of soils (excluding Existing Fill) and (2) soils below 60 feet.

For each soil parameter, a supporting table has been provided to summarize the mean, median, standard deviation, and range of values obtained by soil layer and test type (e.g. CPT, SPT, or laboratory test).

In some cases, soil parameters have been capped at a maximum value. Test results exceeding the maximum value are indicated in red on the histograms.

A5.1 Total Unit Weight

Table A5.1-1
Statistical Summary of Total Unit Weight – Fresno Viaduct

Total Unit Weight	CPT	
	Upper 60 ft	Below 60 ft
No. Tests	1,950	295
Mean, pcf	122	128
Median, pcf	126	132
Standard Deviation, pcf	12	9
Maximum, pcf	137	137
Minimum, pcf	90	94

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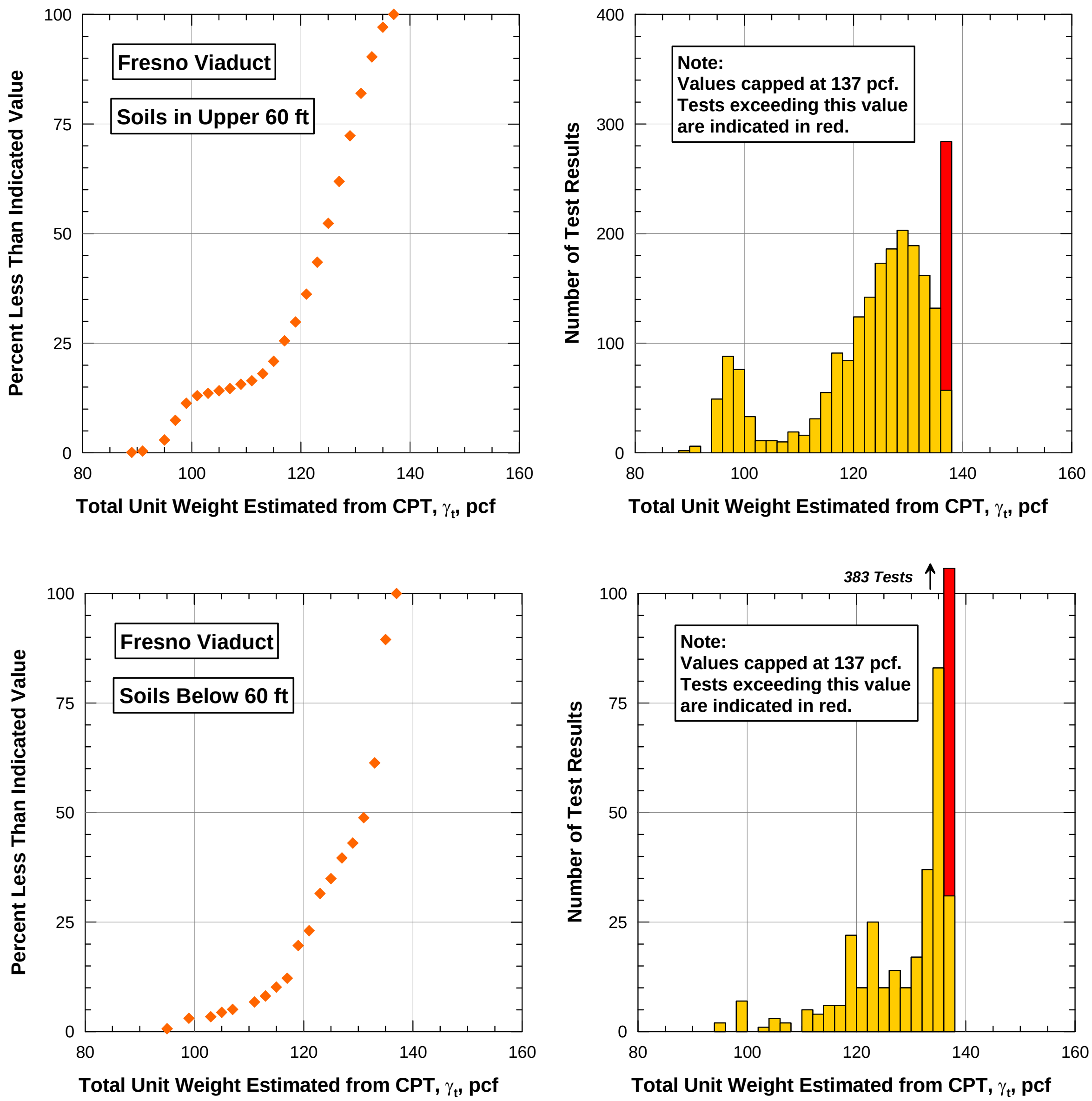

Figure A5.1-1
Statistical Summary of Total Unit Weight Estimated from CPT – Fresno Viaduct

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A5.2 SPT N60

Table A5.2-1
Statistical Summary of SPT N_{60} – Fresno Viaduct

SPT N_{60}	CPT		SPT	
	Upper 60 ft	Below 60 ft	Upper 60 ft	Below 60 ft
No. Tests	2,112	1,009	33	47
Mean, blows/ft	43	60	40	55
Median, blows/ft	41	58	35	55
Standard Deviation, blows/ft	19	17	18	17
Maximum, blows/ft	98	99	95	87
Minimum, blows/ft	3	23	14	24

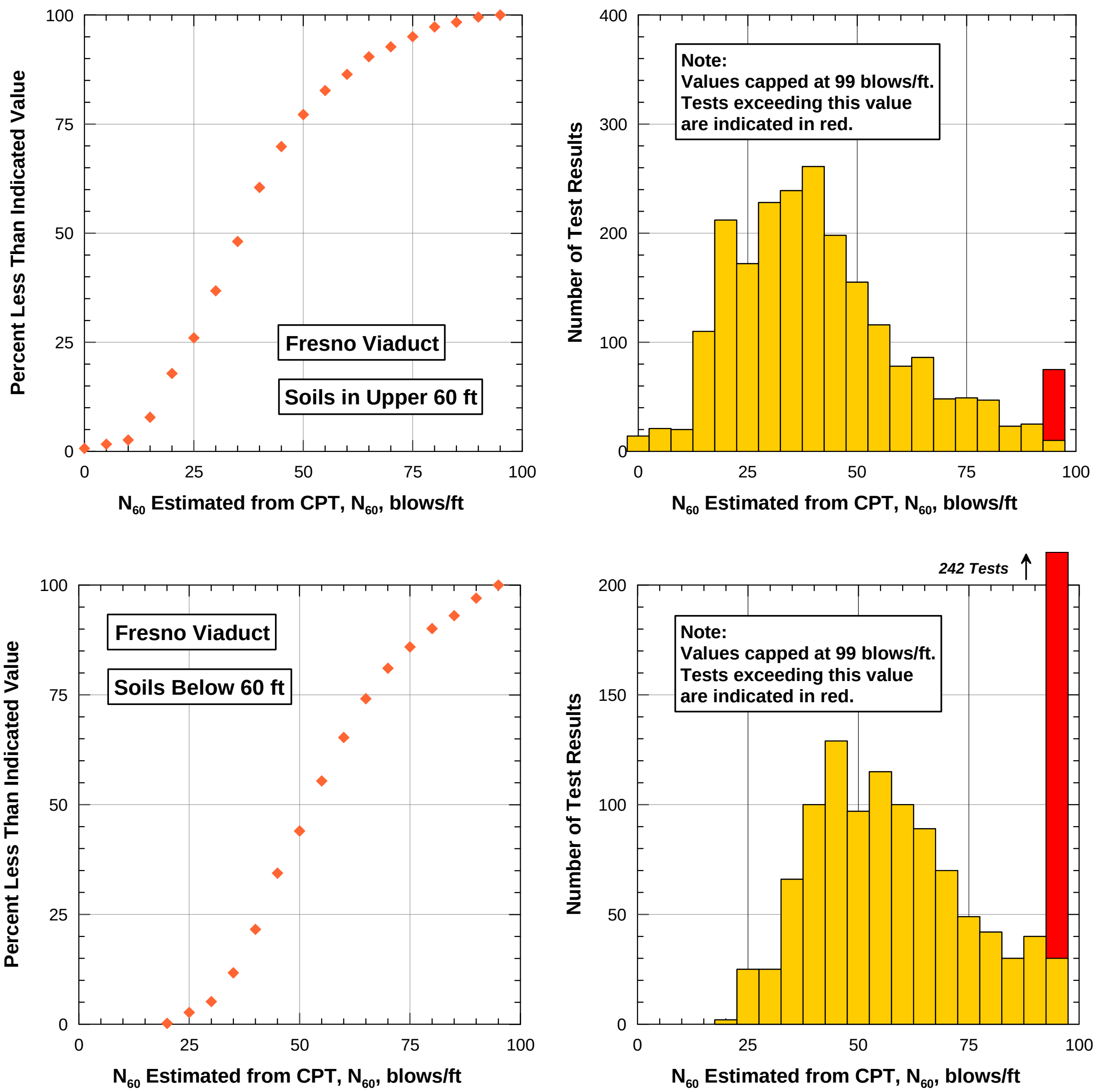

Figure A5.2-1
Statistical Summary of SPT N_{60} Estimated from CPT – Fresno Viaduct

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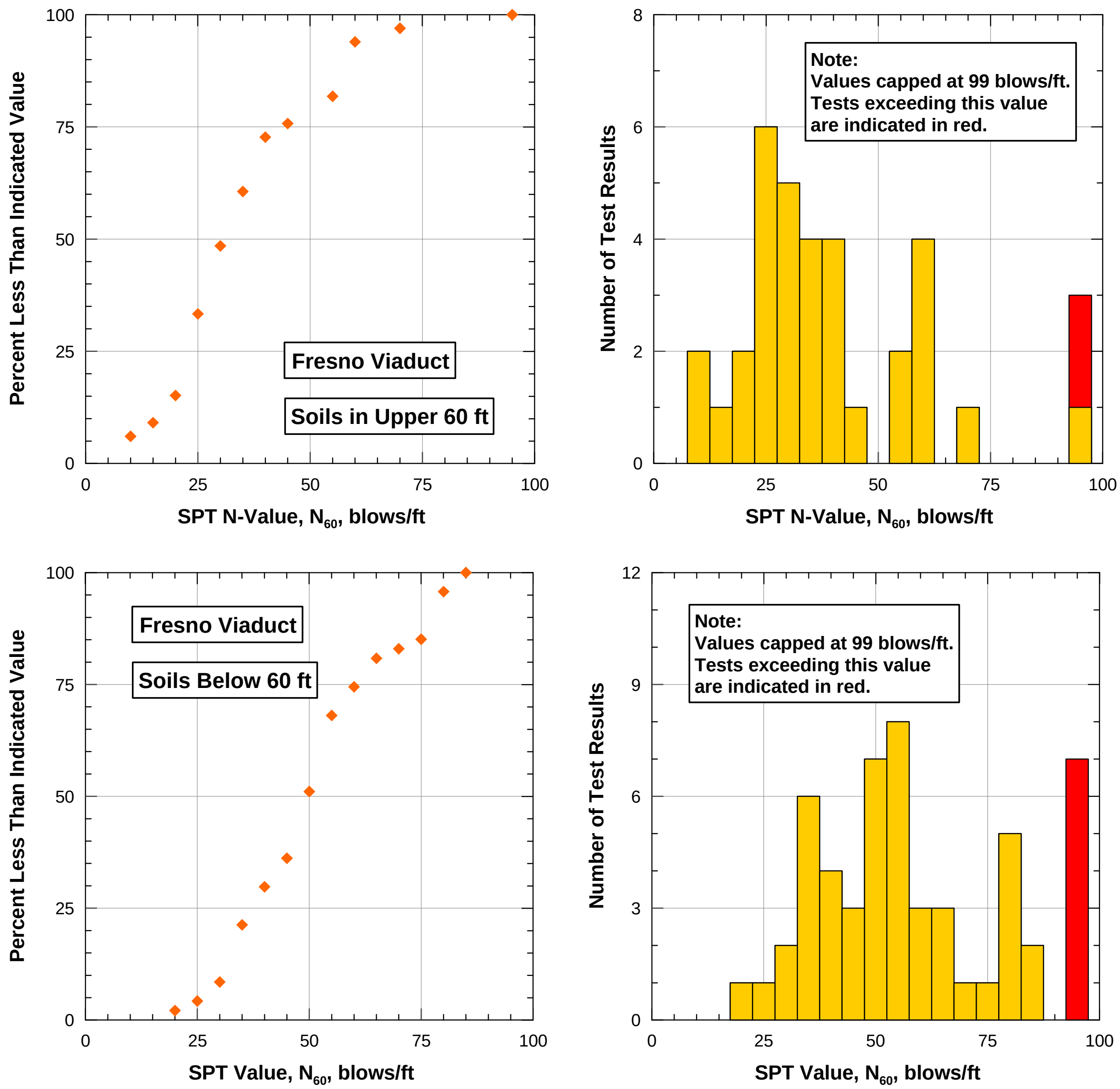

Figure A5.2-2
Statistical Summary of SPT N_{60} – Fresno Viaduct

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A5.3 Effective Friction Angle

Table A5.3-1
Statistical Summary of Effective Friction Angle – Fresno Viaduct

Effective Friction Angle	CPT		SPT		Laboratory	
	Upper 60 ft	Below 60 ft	Upper 60 ft	Below 60 ft	Upper 60 ft	Below 60 ft
No. Tests	2,110	884	19	38	10	13
Mean, deg	43	40	45	44	38	35
Median, deg	43	41	45	45	37	37
Standard Deviation, deg	3	3	3	3	4	4
Maximum, deg	50	49	49	50	43	41
Minimum, deg	32	32	40	38	31	26

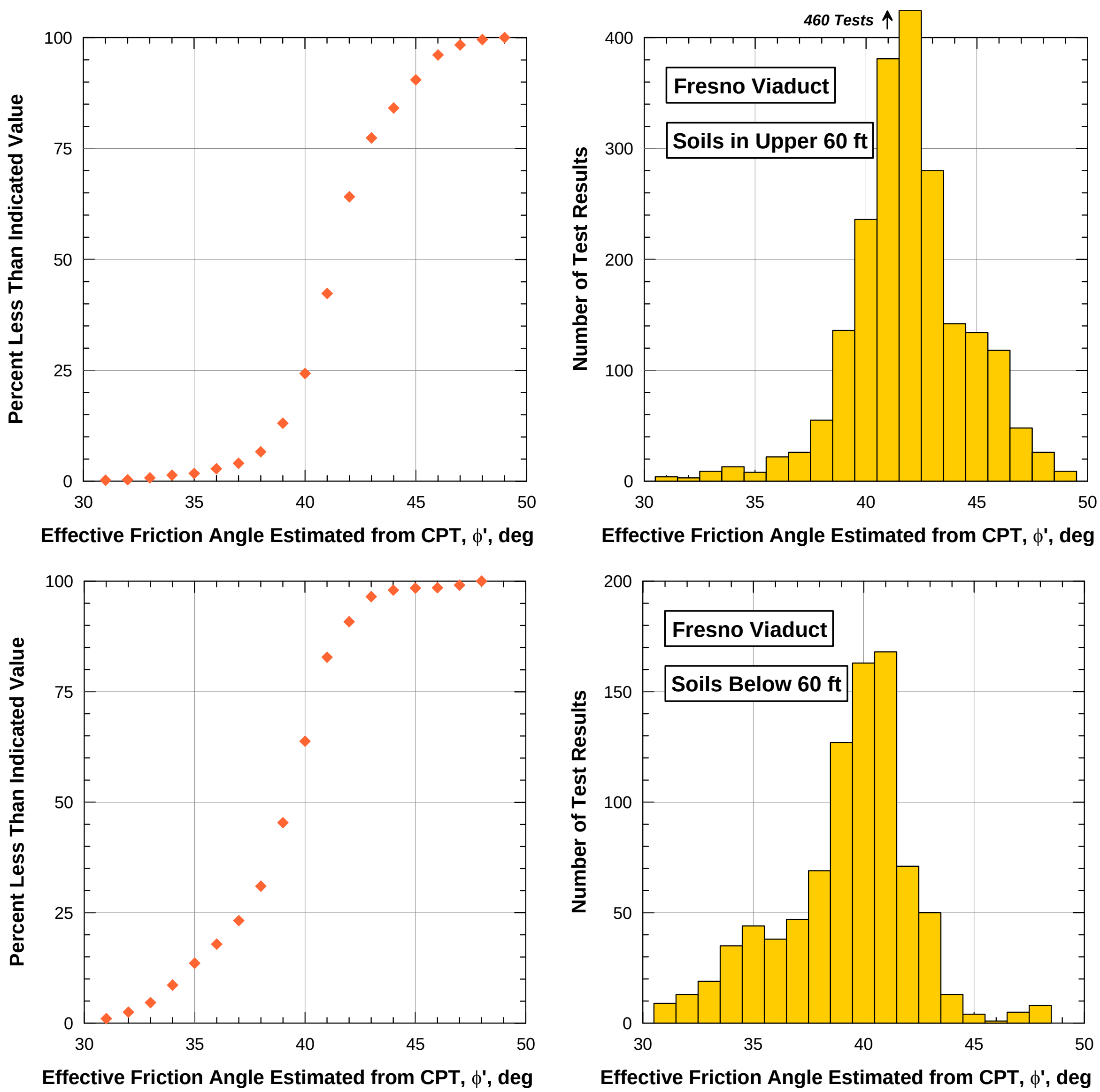

Figure A5.3-1
Statistical Summary of Effective Friction Angle Estimated from CPT – Fresno Viaduct

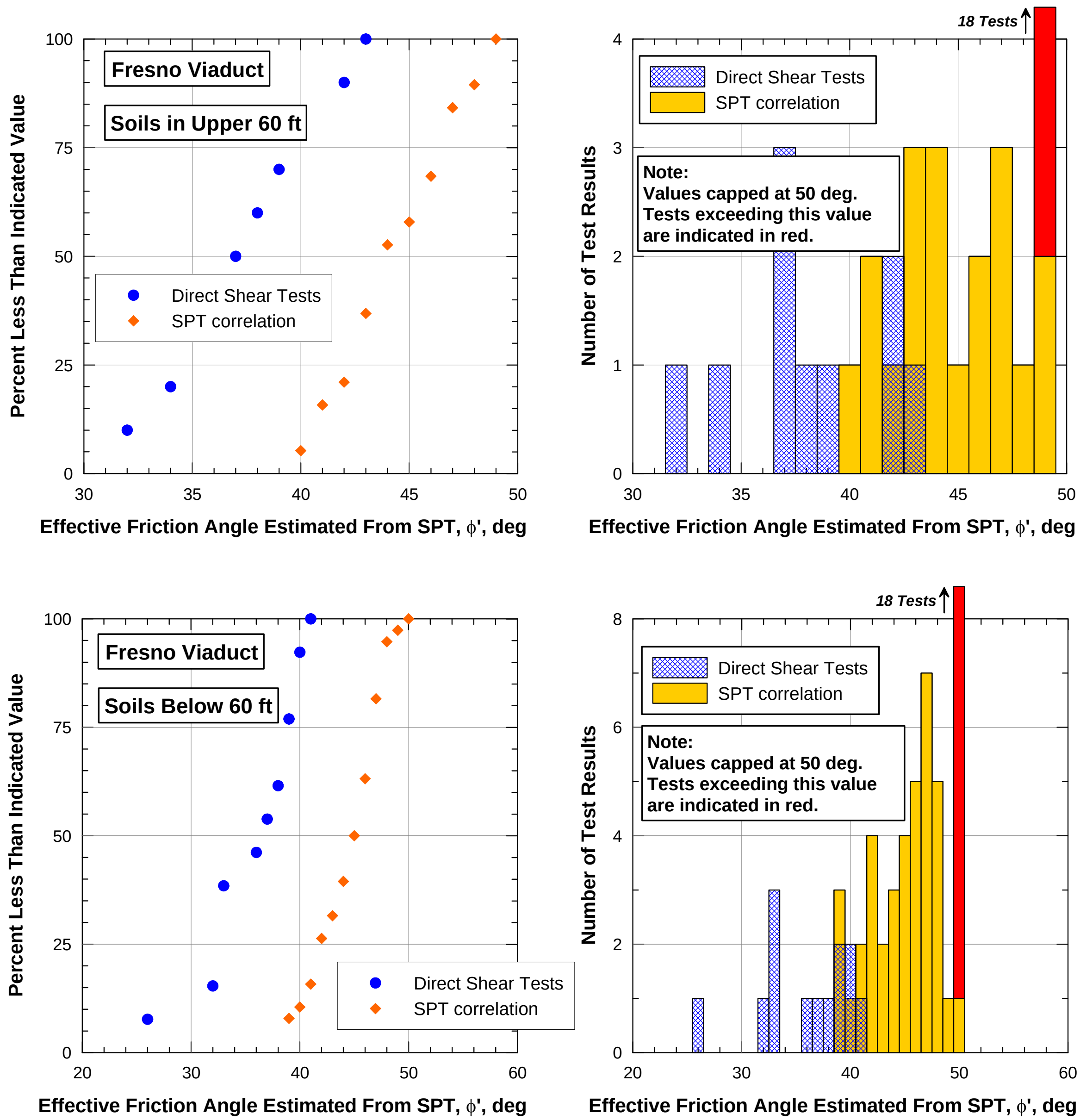

Figure A5.3-2
Statistical Summary of Effective Friction Angle Estimated from SPT – Fresno Viaduct

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A5.4 Cone Tip Resistance

Table A5.4-1

Statistical Summary of Cone Tip Resistance – Fresno Viaduct

CONE TIP RESISTANCE	CPT	
	Upper 60 ft	Below 60 ft
No. Tests	2,329	647
Mean, tsf	234	268
Median, tsf	209	239
Standard Deviation, tsf	136	151
Maximum, tsf	1,630	978
Minimum, tsf	11	50

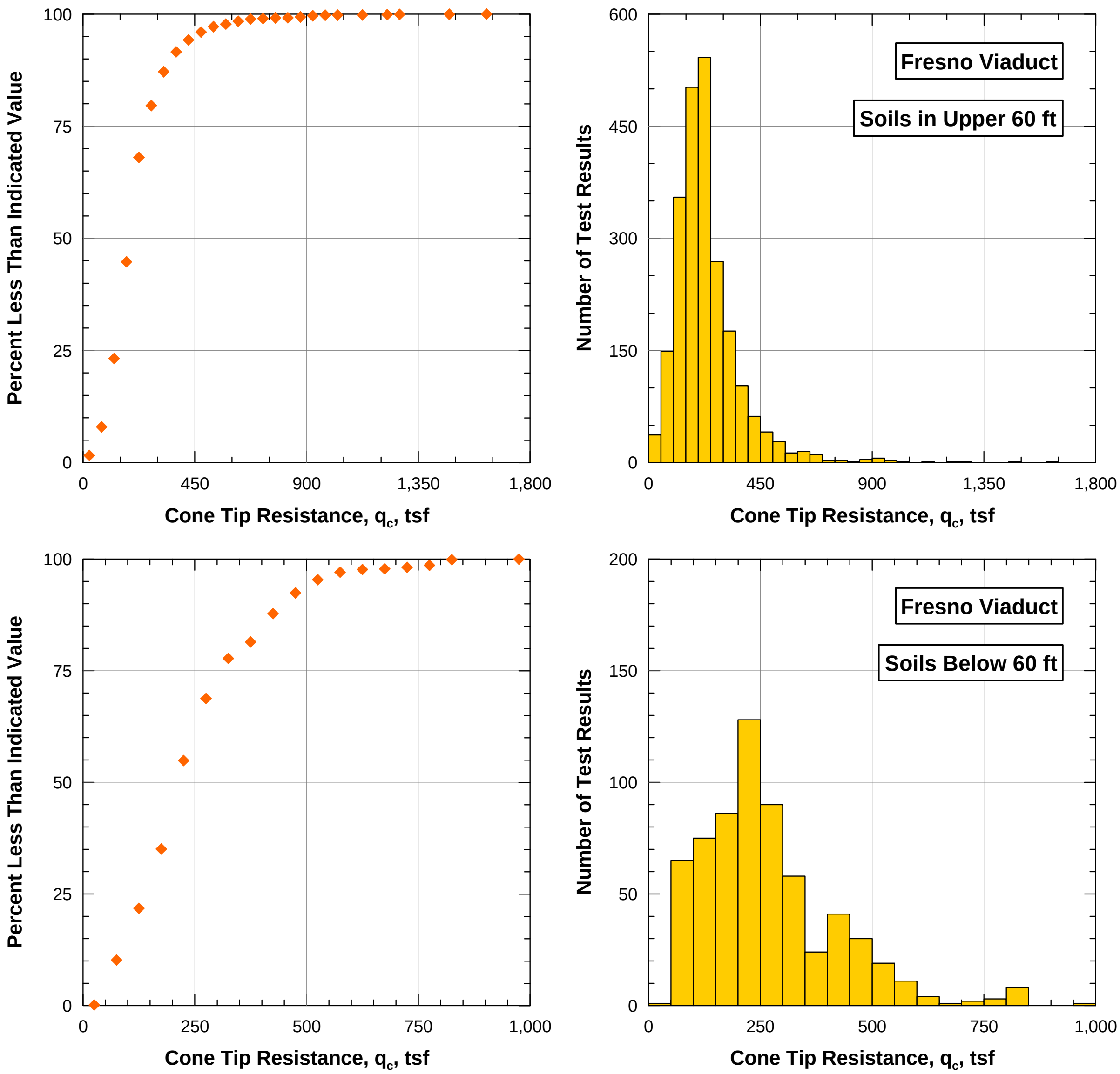

Figure A5.4-1

Statistical Summary of Cone Tip Resistance from CPT – Fresno Viaduct

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A5.5 Soil Modulus

Table A5.5-1

Statistical Summary of Soil Modulus Estimated from SPT– Fresno Viaduct

Soil Modulus	SPT	
	Upper 60 ft	Below 60 ft
No. Tests	28	54
Mean, tsf	354	343
Median, tsf	341	311
Standard Deviation, tsf	111	129
Maximum, tsf	589	653
Minimum, tsf	184	154

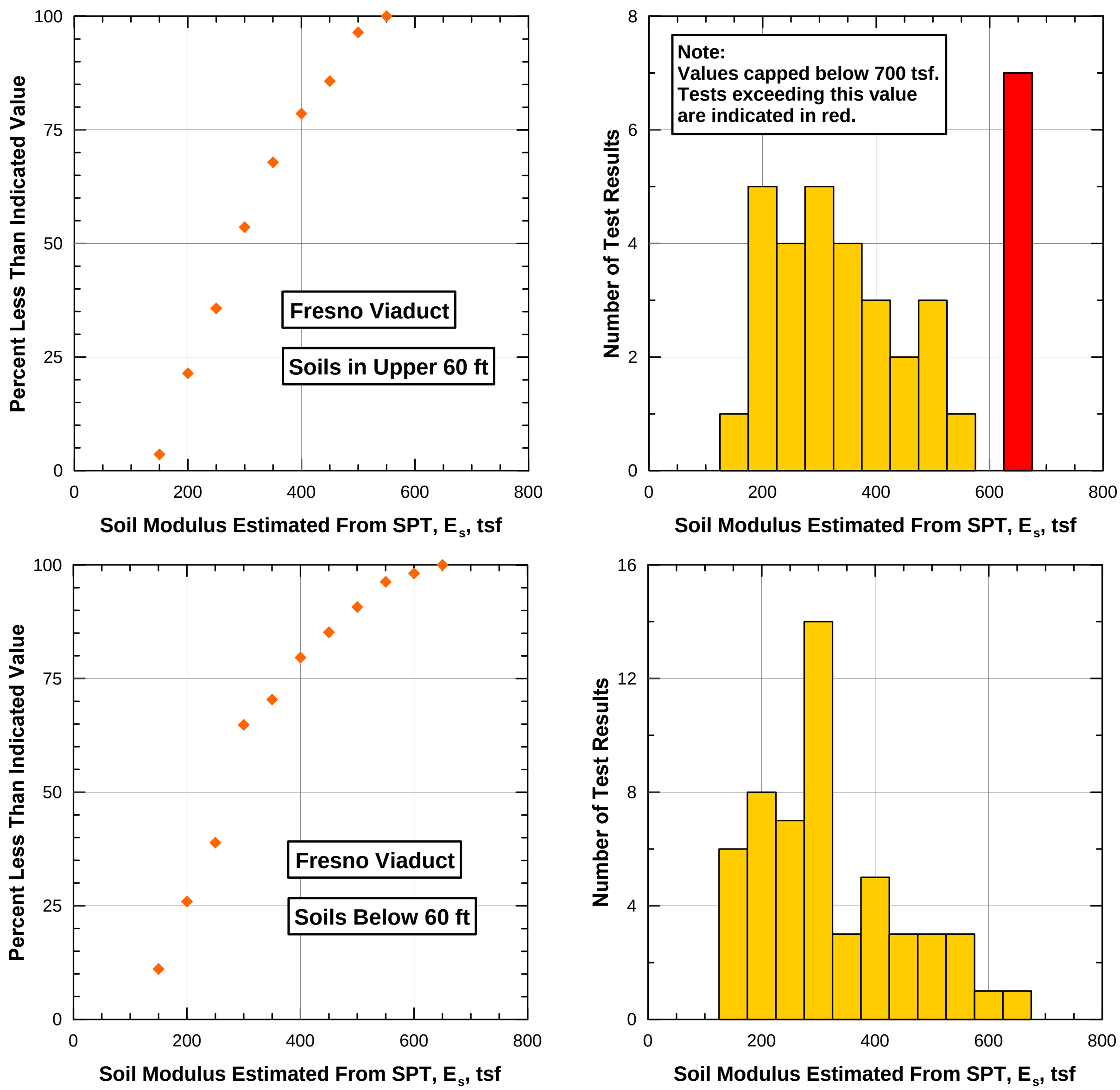

Figure A5.5-1

Statistical Summary of Soil Modulus Estimated from SPT – Fresno Viaduct

Table A5.5-2
Statistical Summary of Soil Modulus Estimated from CPT– Fresno Viaduct

Soil Modulus	CPT	
	Upper 60 ft	Below 60 ft
No. Tests	2,177	1,221
Mean, tsf	464	547
Median, tsf	415	494
Standard Deviation, tsf	274	283
Maximum, tsf	3,260	2,288
Minimum, tsf	22	100

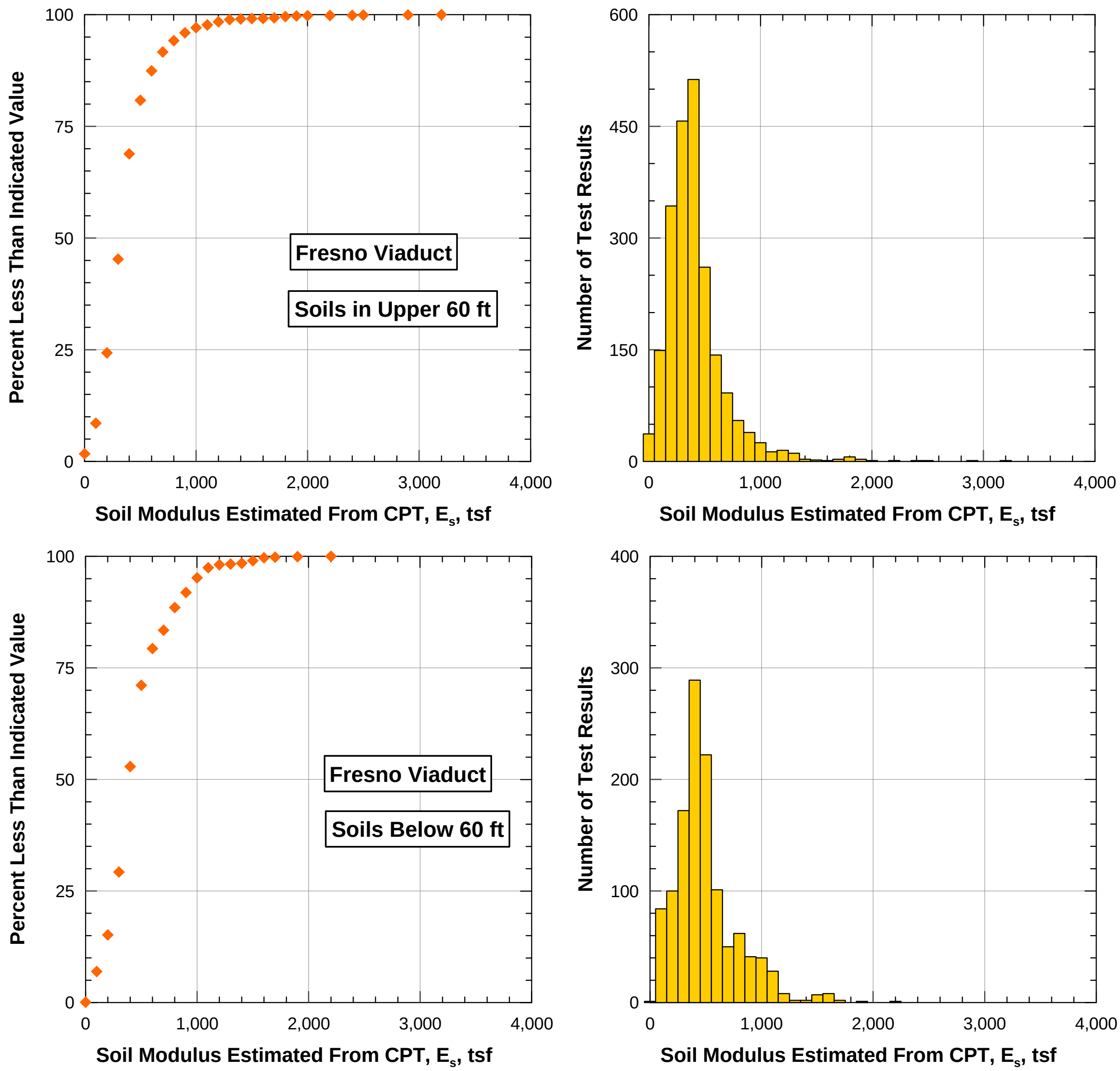

Figure A5.5-2
Statistical Summary of Soil Modulus Estimated from CPT – Fresno Viaduct

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A6.0 At-Grade, Embankment, and Other Ancillary Structures

The following sections present the results of statistical analyses performed on all data obtained from this investigation in the upper 20 feet of soils (excluding Existing Fill). This section is provided as an interpretation of the soil properties that are anticipated for the design and construction of at-grade, embankments, road crossings, pedestrian bridges, and other ancillary structures.

For each soil parameter, a supporting table has been provided to summarize the mean, median, standard deviation, and range of values obtained by soil layer and test type (e.g. CPT, SPT, or laboratory test).

In some cases, soil parameters have been capped at a maximum value. Test results exceeding the maximum value are indicated in red on the histograms.

A6.1 Total Unit Weight

Table A6.1-1
Statistical Summary of Total Unit Weight – Ancillary Structures

Total Unit Weight	CPT
No. Tests	3,383
Mean, pcf	121
Median, pcf	121
Standard Deviation, pcf	10
Maximum, pcf	137
Minimum, pcf	90

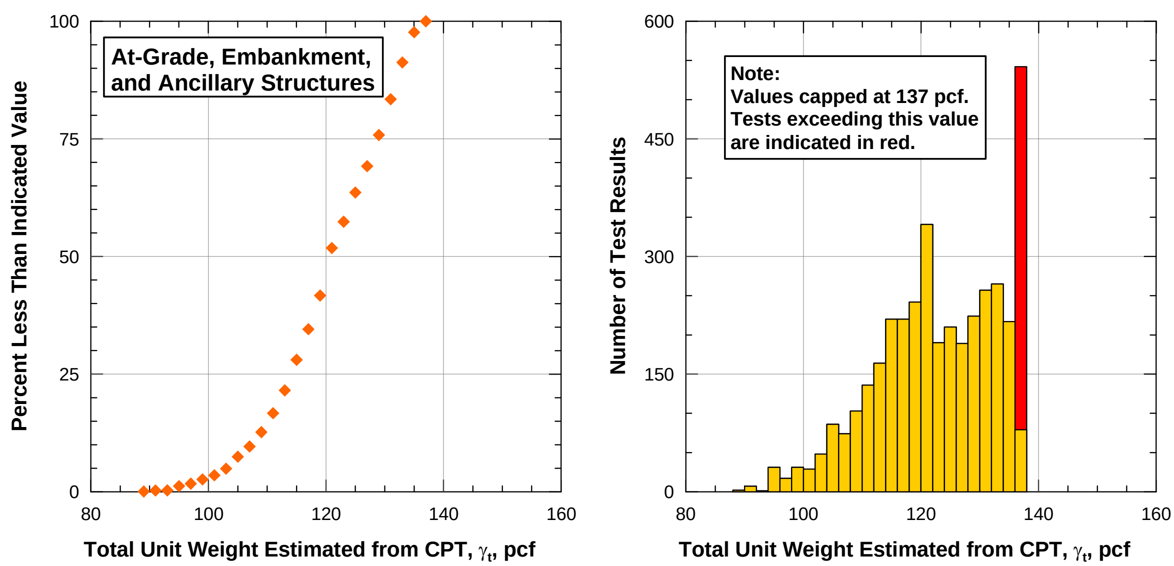

Figure A6.1-1
Statistical Summary of Total Unit Weight Estimated from CPT – Ancillary Structures

A6.2 SPT N_{60}

Table A6.2-1
Statistical Summary of SPT N_{60} – Ancillary Structures

SPT N_{60}	CPT	SPT
No. Tests	3,753	115
Mean, blows/ft	34	35
Median, blows/ft	29	22
Standard Deviation, blows/ft	20	94
Maximum, blows/ft	99	115
Minimum, blows/ft	3	5

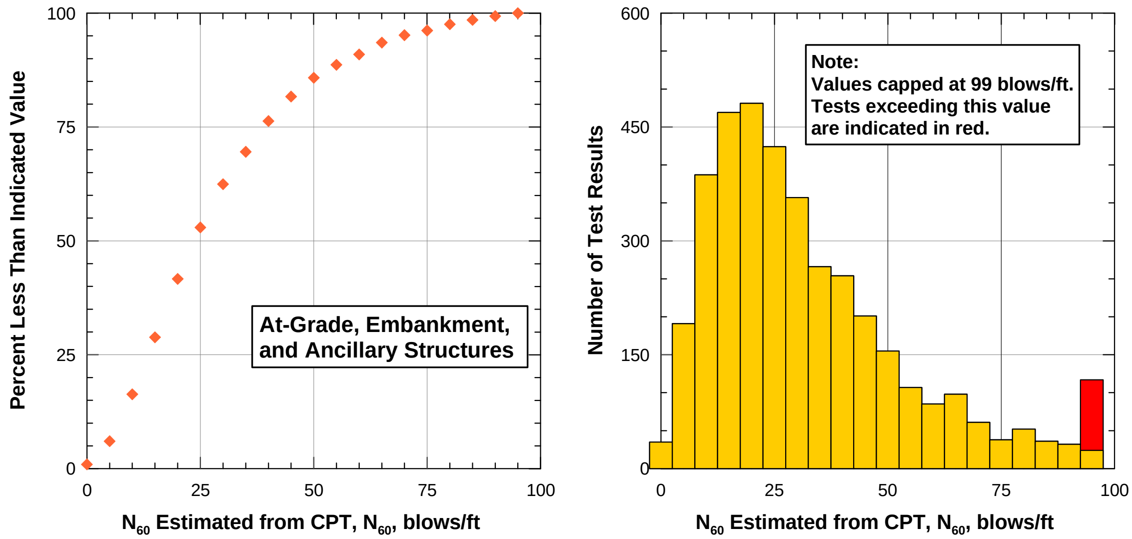

Figure A6.2-1
Statistical Summary of SPT N_{60} Estimated from CPT – Ancillary Structures

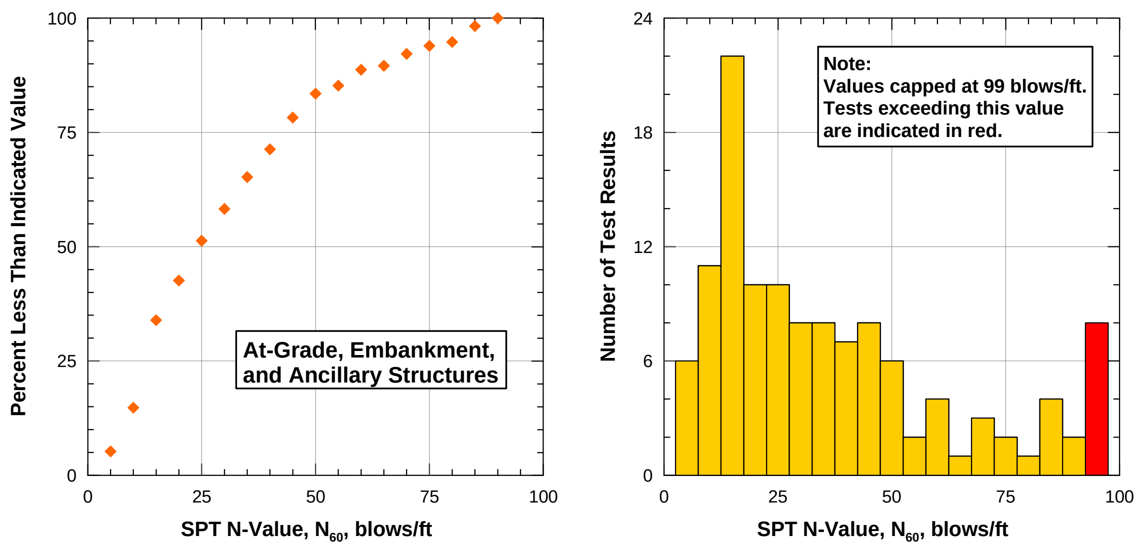

Figure A6.2-2
Statistical Summary of SPT N_{60} – Ancillary Structures

A6.3 Effective Friction Angle

Table A6.3-1

Statistical Summary of Effective Friction Angle – Ancillary Structures

Effective Friction Angle	CPT	SPT	Laboratory
No. Tests	3,742	20	22
Mean, deg	43	42	37
Median, deg	43	4	37
Standard Deviation, deg	3	49	5
Maximum, deg	51	35	43
Minimum, deg	30	20	24

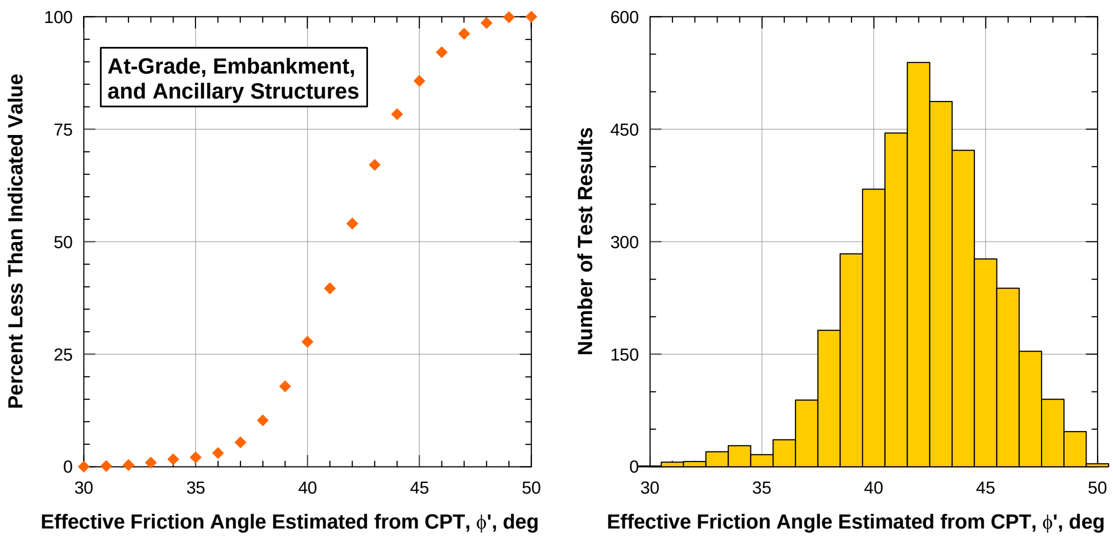

Figure A6.3-1

Statistical Summary of Effective Friction Angle Estimated from CPT – Ancillary Structures

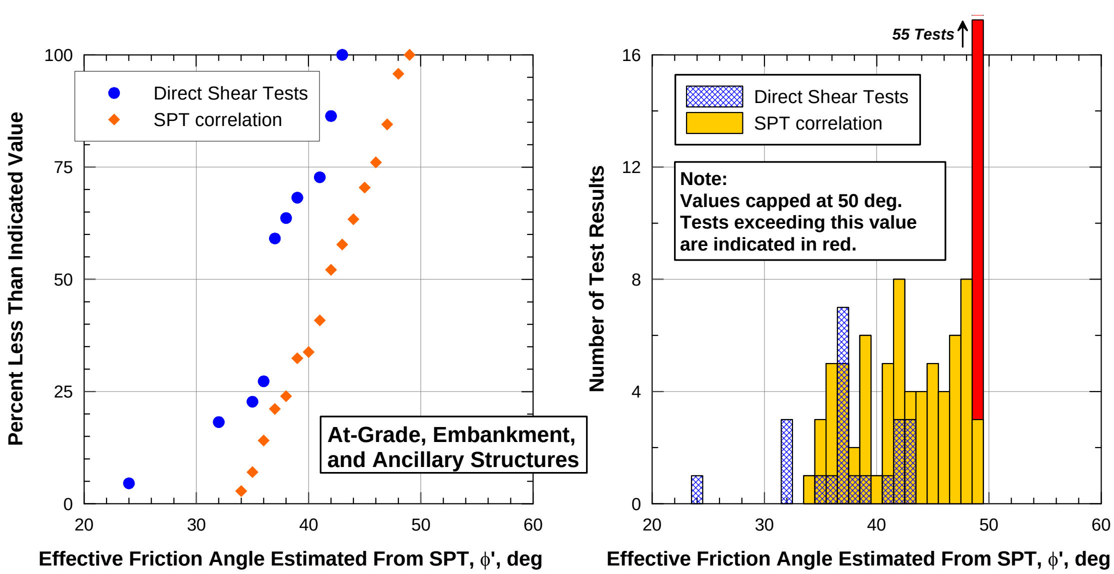

Figure A6.3-2

Statistical Summary of Effective Friction Angle Estimated from SPT – Ancillary Structures

A6.4 Cone Tip Resistance

Table A6.4-1
Statistical Summary of Cone Tip Resistance – Ancillary Structures

Cone Tip Resistance	CPT
No. Tests	3,846
Mean, tsf	178
Median, tsf	146
Standard Deviation, tsf	134
Maximum, tsf	1,630
Minimum, tsf	8

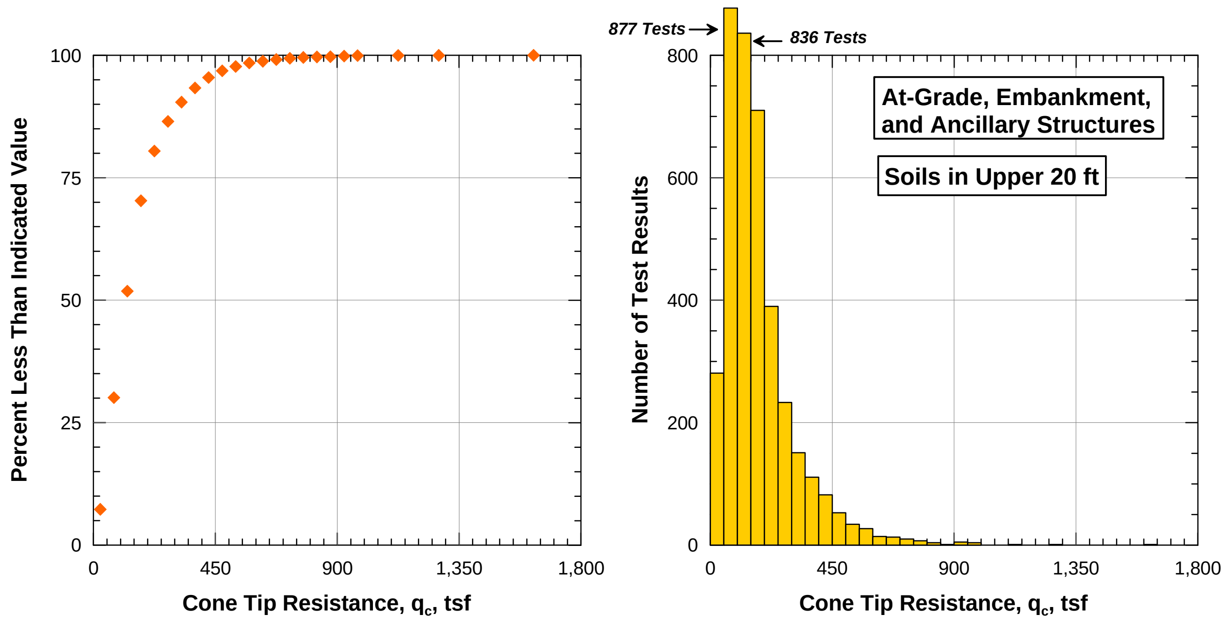

Figure A6.4-1
Statistical Summary of Cone Tip Resistance from CPT – Ancillary Structures

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A6.5 Soil Modulus

Table A6.5-1

Statistical Summary of Soil Modulus Estimated from SPT– Ancillary Structures

Soil Modulus	SPT
No. Tests	75
Mean, tsf	373
Median, tsf	362
Standard Deviation, tsf	145
Maximum, tsf	684
Minimum, tsf	117

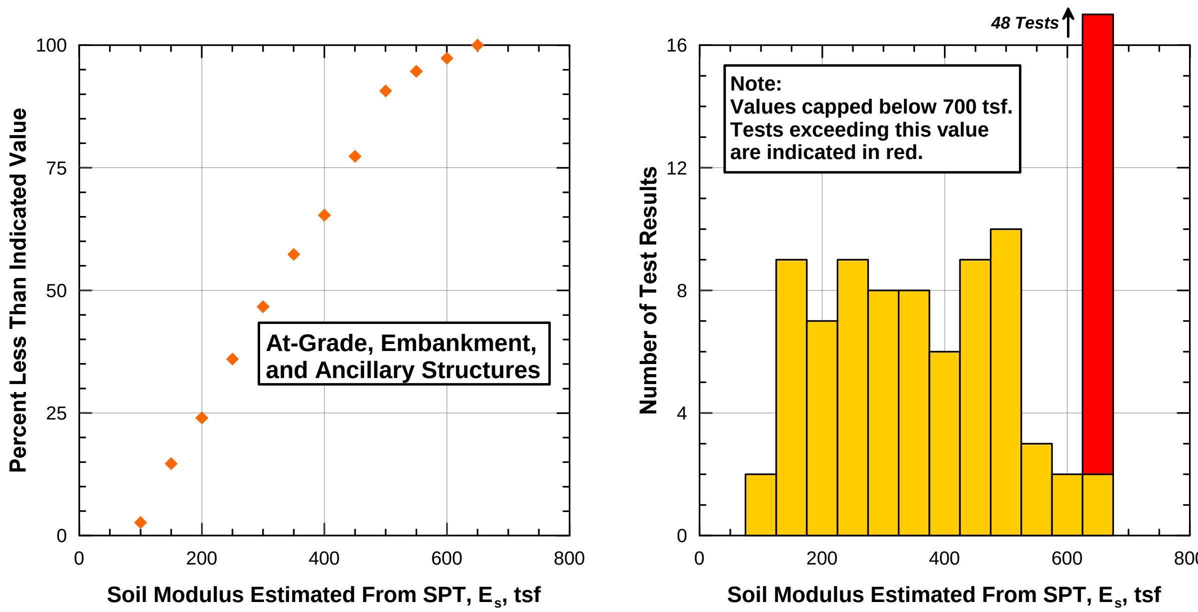

Figure A6.5-1

Statistical Summary of Soil Modulus Estimated from SPT – Ancillary Structures

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Table A6.5-2
Statistical Summary of Soil Modulus Estimated from CPT– Ancillary Structures

Soil Modulus	CPT
No. Tests	725
Mean, tsf	338
Median, tsf	304
Standard Deviation, tsf	225
Maximum, tsf	1,515
Minimum, tsf	33

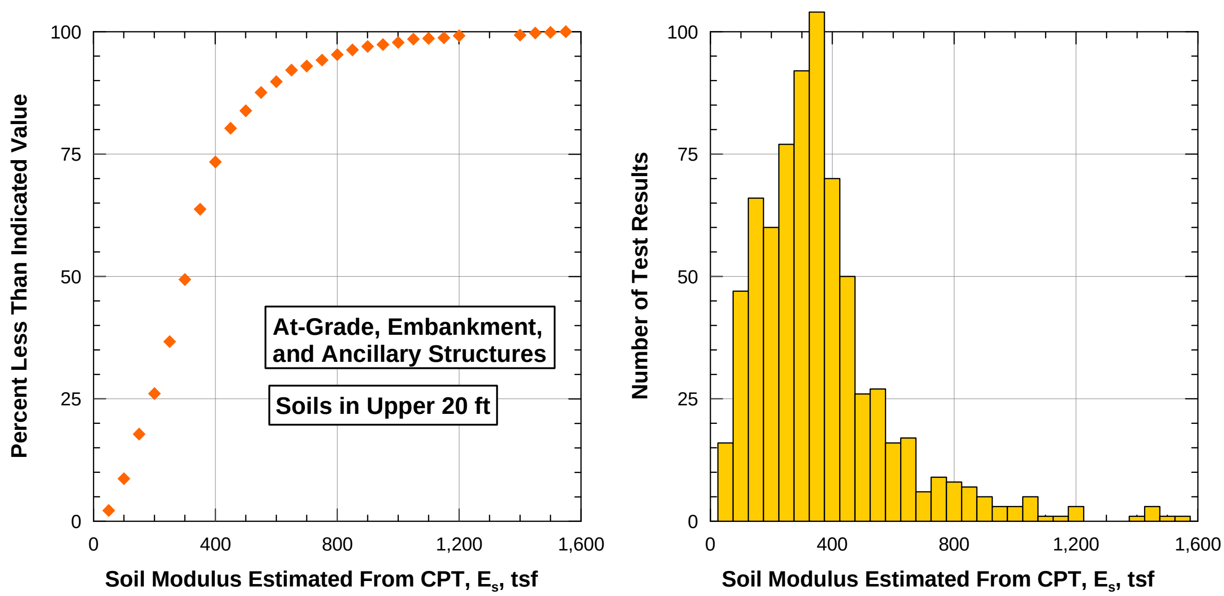

Figure A6.5-2
Statistical Summary of Soil Modulus Estimated from CPT – Ancillary Structures

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A7.0 References

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